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AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

EIGHTEENTH ANNUAL CONVENTION

Held at Cleveland, O.
Feb. 13, 14, 15 and 16, 1922.

VOLUME XVIII

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1922

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BY-LAWS.

ARTICLE I.

MEMBERS.

SECTION 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

SEC. 2. A firm or company shall be treated as a single member.

SEC. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

SEC. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

SEC. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

ARTICLE II.

OFFICERS.

SECTION 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Directors.

SEC. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

SEC. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

Sec. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty, of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

Sec. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

Sec. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

Sec. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President at the first regular session of the Annual Convention.

Sec. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

Sec. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

Sec. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

Sec. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

Sec. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

Article III.

MEETINGS.

Section 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

Sec. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

Sec. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Sec. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

Article IV.

DUES.

Section 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

Sec. 2. The annual dues of each member shall be ten dollars ($10.00).

Sec. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amount of dues for that fiscal year; but he shall not be entitled to a copy of the Proceedings of that year.

Sec. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Institute.

Article V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

Section 1. Proposed Recommended Practice and Specifications to be submitted to the Institute must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter; such Recommended Practice and Specifications shall be considered adopted unless at least 10 percent of the total membership shall vote in the negative.

Article VI.

AMENDMENT.

Section 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE EIGHTEENTH ANNUAL CONVENTION.

Hotel Winton, Cleveland, O.

First Session, Monday, Feb. 13, 1922, 2 p. m.

The convention was called to order by Henry C. Turner, president of the American Concrete Institute.

Report of Committee S-6, on Concrete Roads, was presented by its secretary, C. R. Ege. The report was accepted.

Report of Committee S-2, on Reinforced-Concrete Highway Bridges and Culverts, was presented by its chairman, A. B. Cohen. The report was accepted.

Moving pictures of roadbuilding construction were presented by George A. Sherron of the Koehring Machine Co.

President H. C. Turner presented a paper on building conditions in England.

Second Session, Monday, Feb. 13, 1922, 8 p. m.

President H. C. Turner in the chair.

Report of Committee S-5, on Reinforced-Concrete Houses, was presented by its secretary, A. C. Irwin.

Director E. D. Boyer in the chair.

J. A. Ferguson, member of Committee S-5, presented a discussion of the committee report.

Mr. A. E. Lindau in the chair.

There was further discussion of the report of Committee S-5 and the report was then accepted by the Institute as a progress report.

The following papers on concrete houses were read:

"Concrete Houses at Small Cost," by Miss Kate Gleason.

"Gun-Stone Houses at Watertown, Mass.," by H. Whittemore Brown.

"Reinforced-Concrete Houses," by B. V. Hueber.

"Concrete Houses," by P. R. Smith.

Third Session, Tuesday, Feb. 14, 1922, 9.30 a. m.

President H. C. Turner in the chair.

Report of Committee E-3, on Research, was read by its chairman, Prof. W. K. Hatt. The report was accepted.

The following papers were read and discussed:

"Flexural Strength of Plain Concrete," by Duff A. Abrams.

"Ball Test Applied to Concrete," by W. K. Hatt.

"Fatigue Tests of Concrete," by W. K. Hatt.

The secretary read the report of the Board of Direction.

The president appointed a Resolutions Committee and tellers for the counting of the ballots.

FOURTH SESSION, TUESDAY, FEB. 14, 1922, 2 P. M.

Vice-President W. P. Anderson in the chair.

The following papers were read and discussed:

"The Limitation and Possibilities of the Present and Future Markets for Concrete Building Units," by W. D. M. Allan.

"The Use of Concrete Block in the Construction of Houses for Employees of the Youngstown Sheet & Tube Co.," by B. E. Brooke.

Report of Committee P-1, on Standard Concrete Building Units, was presented by its secretary, W. R. Harris.

The minority report of Committee P-1 was presented by S. H. Wightman.

A discussion of the two reports was offered by John W. Lowell, a member of the committee. Mr. Lowell's compromise suggestions were discussed.

A defense of the majority report was presented by the chairman of the committee, R. F. Havlik. It was moved and seconded that the majority and minority reports be printed and that the committee be asked to get together to present a majority report at the next annual meeting. The motion was adopted by a majority vote.

Report of Committee P-5, on Fire Resistance of Concrete Building Units, was presented by its chairman, Leslie H. Allen. The report was accepted and the resolution noted in the report was adopted by the meeting.

Report of Committee P-4, on Concrete Staves, was read by Mr. Jackson. The report was accepted.

FIFTH SESSION, TUESDAY, FEB. 14, 1922, 8 P. M.

President H. C. Turner in the chair.

Report of Committee J-2, on Concrete Pipe, was presented by its chairman, B. S. Pease. The report was received.

Report of Committee P-7 on Concrete Pipe, Drain Tile and Conduit was presented by its chairman, C. F. Buente. The report was received and the recommendation of the committee that certain standards of the American Society for Testing Materials be adopted as standards of the Institute was confirmed.

Report of Committee P-6, on Products Plant Operation, was presented by its chairman, S. H. Wightman. The report was received.

Report on Committee S-1, on Concrete Chimneys, was presented by its chairman, Eric Plagwit. The report was received.

SIXTH SESSION, WEDNESDAY, FEB. 15, 1922, 9.30 A. M.

President H. C. Turner in the chair.

A demonstration of practical field problems was given by Prof. Duff A. Abrams and Stanton Walker of the Structural Materials Research Laboratory, Lewis Institute, Chicago.

Report of Committee C-1, on Contractors Plant, was presented by John G. Ahlers, member of the committee. The report was accepted.

Report of Committee C-2, on Concrete Floor Finish, was presented by its chairman, N. M. Loney. The report was accepted.

The following papers were read and discussed:

"Concrete Floor Surfaces," by Joseph C. Grady.

"Bailey Floor Finish," by W. M. Bailey (read by title only).

The Wason Medal for the most meritorious paper presented to the 1921 Convention was presented by A. R. Lord of the Wason Medal Committee to Prof. H. M. Westergaard of the University of Illinois for his paper entitled "Analysis of Homogeneous Elastic Plates," a part of the joint paper on "Moments and Stresses in Slabs" presented by Mr. W. A. Slater and Prof. H. M. Westergaard.

The tellers announced the election of the following officers:

President; William P. Anderson, president, Ferro Concrete Construction Co., Cincinnati, Ohio.

Vice-president (full two-year term); M. M. Upson, vice-president, Raymond Concrete Pile Co., New York City.

Vice-president (one-year term); A. E. Lindau, vice-president, Corrugated Bar Co., Buffalo, N. Y.

Treasurer; Harvey Whipple, Editor, "Concrete," Detroit, Mich.

Directors, First District, Charles E. Nichols, Stone & Webster Co., Boston, Mass.; *Second District*, E. D. Boyer, Atlas Portland Cement Co., New York City; *Sixth District*, Arthur Bent, Bent Brothers, Contractors, Los Angeles, Cal.

President Anderson was escorted to the chair by past president Richard L. Humphrey and addressed the meeting.

SEVENTH SESSION, WEDNESDAY, FEB. 15, 1922, 2 P. M.

President H. C. Turner in the chair.

Vice-president elect A. E. Lindau in the chair.

Report of Committee C-3, on Concrete Surfaces, was presented by its chairman, J. C. Pearson. The report was accepted.

Report of Committee E-7, on Waterproofing, was presented by J. H. Libberton, secretary. The report was accepted.

Report of Committee C-4, on Metal Forms, was presented by W. F. Lockhardt, of the committee. The report was accepted.

The following papers were read and discussed:

> "Corrosion of Steel Reinforcing," by M. E. Hibbs, construction engineer, Bureau of Building Inspection, Philadelphia.
>
> "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gages," by Prof. George E. Beggs, Princeton University.

EIGHTH SESSION, THURSDAY, FEB. 16, 1922, 9 A. M.

President H. C. Turner in the chair.

The session was devoted to a discussion of the report of the Joint Committee on Concrete and Reinforced Concrete.

NINTH SESSION, THURSDAY, FEB. 16, 1922, 2.30 P. M.

The session was devoted to a further discussion of the report of the Joint Committee on Concrete and Reinforced Concrete.

THE WASON MEDAL.

Awarded Each Year to the Author of the Most Meritorious Paper Presented to the Previous Annual Convention.

Awarded 1921 to

H. M. Westergaard, for his paper, "Analysis of Homogeneous Elastic Plates."

Previous Awards.

1916—A. B. McDaniel, "Influence of Temperature on the Strength of Concrete."

1917—Charles R. Gow, "History and Present Status of the Concrete Pile Industry."

1918—Duff A. Abrams, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919—W. A. Slater, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920—W. A. Hull, "Fire Tests of Concrete Columns."

Papers Read Before the 18th Annual Convention of the American Concrete Institute

FLEXURAL STRENGTH OF PLAIN CONCRETE

By Duff A. Abrams *

The flexural strength of concrete is low as compared with its compressive strength; the use of steel bars to supply this deficiency is the basic principle of reinforced concrete construction. For many years little attention has been given to the flexural strength of concrete, due to the practice of disregarding the tensile stresses in the concrete in the design of *reinforced*-concrete members and structures. With the advent of concrete roads and pavements, the flexural strength of concrete again became important; it may in fact prove the determining factor in working out a rational and economical design of slabs for this purpose. Concrete roads have presented many new engineering problems, one of which is the design of a comparatively thin slab to carry heavy rolling loads. Critical tensile stresses may occur in any direction, in either the top or bottom surface; the span length is uncertain on account of the indeterminate nature of the support.[1] To provide adequate steel reinforcement would involve a prohibitive expense, consequently engineers have come to realize that primary dependence must be placed on the ability of the plain concrete to develop the flexural strength necessary for a proper distribution of the load over the subgrade. The exact thickness of slab necessary for this purpose cannot now be fixed by mathematical analysis, but must be determined as a result of experience. Considerable advance toward a rational design of road slabs may be expected to result from the studies of subgrade soils, the effect of moisture and temperature on the slab and the effect of impact that are now being carried out by the U. S. Bureau of Public Roads and a number of the state highway departments.

Flexural tests of mortar and concrete have been reported by Feret,[2] Fuller and Thompson,[3] Sabin[4] and others, but it seemed desirable to review the subject in the light of recent developments which have led to a better understanding of the factors which enter into the scientific design of concrete mixtures, and a better appreciation of the necessity for exact control of the concrete in carrying out studies of this kind.

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*Professor in Charge, Structural Materials Research Laboratory, Lewis Institute, Chicago.

[1] Subgrade Support and Pavement Design, by Clifford Older, Eng. News-Rec., v. 86, p. 210, Feb. 3, 1921.

[2] Etude Experimentale du Ciment Arme, by R. Feret; p. 576, 1906. (See Fig. 13 below.)

[3] Laws of Proportioning Concrete, by Fuller and Thompson; Trans. Am. Soc. Civil Eng., v. 59, p. 67, 1907. (See Fig. 12 below.)

[4] Cement and Concrete, by L. C. Sabin, 1907.

The tensile resistance is the limiting factor in flexural tests of plain concrete, yet a careful distinction must be made between *tension* and *flexural* stresses. Flexural stress here refers to the type of stress developed by beam action. Due to the curvilinear relation between deformation and stress in concrete[5] and other non-homogeneous materials, the tensile stress developed in flexure, when computed by the usual methods, far exceeds the resistance of the material in pure tension. The computed unit tensile strength at which failure occurs is termed the "Modulus of Rupture" of the material. (Tests by Mills[6] showed that *tensile strength* of concrete is probably not more than 8 to 10% of its compressive strength in 6-in. cubes, whereas the tests in this report show the modulus of rupture to be 20% or more of the compressive strength.) Pure tension tests of concrete are difficult to make and may give misleading results on account of non-uniform distribution of stress across the section.

While it is sound practice to disregard the flexural resistance of the *concrete* in *reinforced* concrete members, it should be noted that load tests on bridges[7] and buildings[8] have shown the concrete to contribute in a marked degree to increasing the load-carrying capacity of the structure.

The tests covered by this report were made as a part of a general investigation of concrete and concrete materials being carried out through the cooperation of Lewis Institute and the Portland Cement Association, at the Structural Materials Research Laboratory, Chicago.

The discussion of these tests is followed by a brief review of the relation of modulus of rupture to compressive strength of a number of other materials of construction.

SCOPE OF TESTS.

750 concrete beams were tested by loads applied at the ⅓ points of a 36-in. span. In general, the beams were of plain concrete 10 in. wide, 7 in. deep. In one group of tests the depth of the beam was varied from 4 to 10 in.; in another group 1 to 8⅜-in. round reinforcing bars were used. In general the aggregates consisted of sand and pebbles; in one group of tests other types of coarse aggregates were used. Most tests were made at 28 days; in certain groups tests were made at ages of 7 days to 1 year.

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[5] Modulus of Elasticity of Concrete, by Stanton Walker. Proc. Am. Soc. Testing Mat., 1919, Part II.

[6] Materials of Construction, by A. P. Mills, 1915, p. 192. (In stating these percentages certain allowance has been made for discrepancy in ages of specimens tested by Mills.)

[7] Test of a 40-ft. Reinforced Concrete Highway Bridge, by Duff A. Abrams. Proc. Am. Soc. Testing Mat., 1913, p. 884.

[8] Moments and Stresses in Slabs, by Westegaard and Slater, Proc. Am. Concrete Inst., 1921, p. 415, see bibliography, p. 533.

The following topics were studied in this series:—

1. Position of beam during test,
2. Quantity of cement,
3. Quantity of mixing water,
4. Size of aggregate,
5. Grading of aggregate,
6. Curing condition of concrete,
7. Depth of beam,
8. Kind of aggregate,
9. Age of concrete,
10. Relation between flexural and compressive strength,
11. Relation between strength and water-ratio of concrete,
12. Number of steel reinforcing bars,
13. Uniformity of tests.

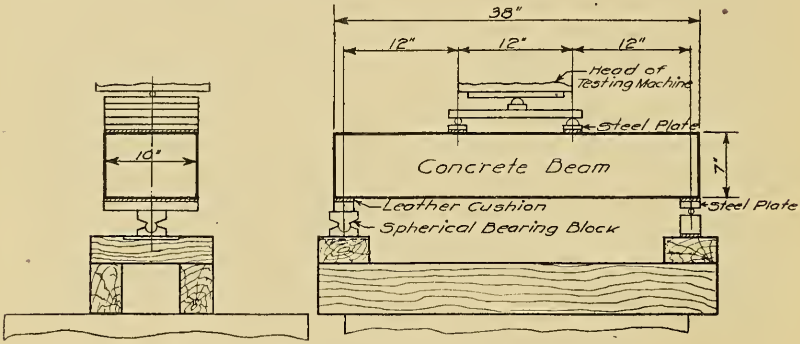

FIG. 1.—METHOD OF LOADING CONCRETE BEAMS.

An important feature of the report is the comparisons between flexural and compressive strength of concrete from the same batches. 750 tests of 6 x 12-in. cylinders are included.

MATERIALS.

The cement consisted of a mixture of equal parts of 4 brands of portland cement purchased in Chicago; complete data of tests are given in Table 1.

Sand from Janesville, Wis., and pebbles from Elgin, Ill., were used in most of the tests. In one group crushed stone and slag were used as coarse aggregate. Sieve analyses and miscellaneous data concerning the aggregates will be found in Table II. The sieves used were of square mesh wire cloth recommended by the American Society for Testing Materials.[9]

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[9] Tentative Method of Test for Sieve Analysis of Aggregate for Concrete, Proc. A. S. T. M., 1921.

The sieve analysis curves for aggregates (Fig. 2) are platted in terms of the percentages coarser than each of the sieves in the set, using as abscissae the logarithms of the clear openings in inches. Since the clear opening of each sieve is just double the width of the next smaller sieve, the logarithmic scale consists simply of a series of equi-distant ordinates. Special paper is not required for a logarithmic scale for this purpose. The fineness modulus[10] of the aggregate is a useful measure of its size and grading; it is the sum of the percentages in the sieve analysis expressed as above, divided by 100. Graphically it is proportional to the area under the sieve analysis curves in Fig. 2. A well-graded concrete sand has a fineness modulus of 2.25 to 3.25; a coarse aggregate graded No. 4 to 1½ in. has a value of about 7.00.

TABLE I.—TESTS OF PORTLAND CEMENT.

The cement used in all tests consisted of a mixture of equal parts of four brands purchased in Chicago.

Tests were made in accordance with Standard Specifications and Tests for Portland Cement of American Society for Testing Materials.

MISCELLANEOUS TESTS.

Cement Lot No.	Fineness, Residue on No. 200 Sieve.	Normal Consistency, per cent by weight.	Time of Setting.				Soundness Test (over boiling water).
			Vicat Needle.		Gillmore Needle.		
			Initial.	Final.	Initial.	Final.	
			h. m.	h. m.	h. m.	h. m.	
5219	18.8	24.0	3 40	8 20	5 45	9 40	OK
5259	17.6	23.0	3 45	8 00	6 30	8 30	OK

MORTAR STRENGTH TESTS.

1 : 3 Standard Sand Mortar.

Values for strength are given in pounds per square inch.

Cement Lot No.	Mixing Water per cent.	Tensile Strength of Briquets.					Compressive Strength of 2 x 4-in. Cylinders.				
		7 d.	28 d.	3 m.	6 m.	1 y.	7 d.	28 d.	3 m.	6 m.	1 y.
5219	10.5	235	365	425	380	405	1670	2570	3520	4250	3840
5259	10.3	280	430	410	385	355	1720	2870	3710	4150	4370

TEST PIECES.

In general flexural tests were made on plain concrete beams 10 in. wide, 7 in. deep and 38 in. long. In one group of tests the depth was varied from 4 to 10 in. In another group one to eight ⅜-in. round bars were used as reinforcement. Parallel compression tests were made on 6 x 12-in. cylinders from the same batches.

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[10] For further details see "Design of Concrete Mixtures," by Duff A. Abrams, Bull. 1, Structural Materials Research Laboratory, Lewis Institute, Chicago.

The quantities of aggregate required for each batch were determined by weight on the basis of the unit weight of the aggregates mixed as used. The quantity of water for a batch was measured by means of a calibrated tank. All mixtures were by volume; that is, one volume of cement to a

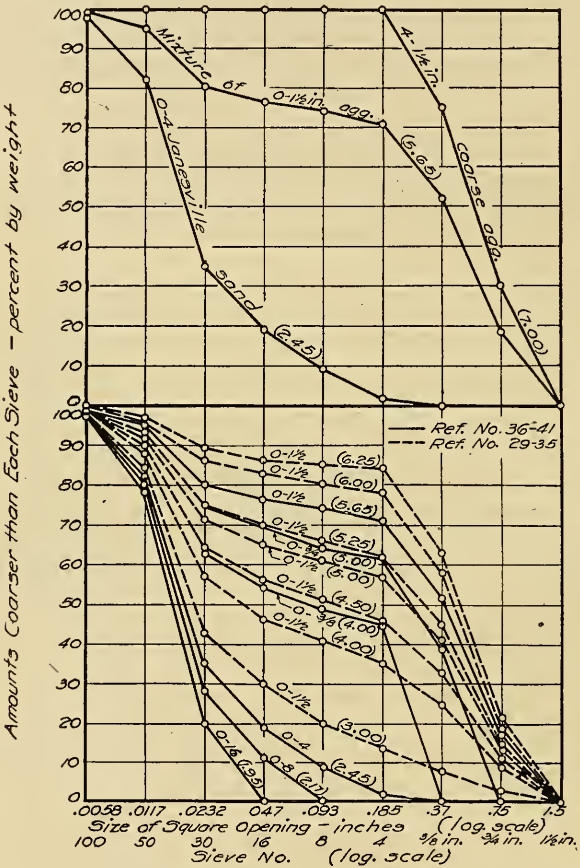

FIG. 2.—SIEVE ANALYSIS CURVES OF AGGREGATES USED IN TESTS.

Details are given in Table II.

given number of volumes of aggregate were mixed together. The mixtures are expressed as 1: 6, 1: 4, etc. regardless of the size or grading of the aggregate. One sack of cement (94 lb.) was considered 1 cu. ft. The consistency or workability of the concrete was measured by the "flow table"[11]. Concrete with relative consistency 1.00 gave a flow of about 180; other relative consistencies were based on proportional quantities of mixing water. A relative consistency of 1.10 gave a flow of about 220.

The concrete was mixed in a Smith "Mascot" mixer of 3½ cu. ft. capacity. Sufficient concrete was mixed to make 2 beams and 2 cylinders. Each batch was mixed 1 minute after all materials were in the machine.

TABLE II.—SIEVE ANALYSIS AND UNIT WEIGHT OF AGGREGATES.

Square mesh wire cloth sieves manufactured by The W. S. Tyler Co., Cleveland, O., were used in making sieve analyses. Each sieve has a clear opening twice the width of the preceding.

Sieve analysis curves are platted in Fig. 2.

Ref. No.	Aggregate.		Sieve Analysis. Amount Retained on Each Sieve, per cent by Weight.									Fineness Modulus of Aggregate.*	Unit Weight, lb. per cu. ft.
	Kind.	Size.	100	50	30	16	8	4	⅜	¾	1½		
36	Janesville sand....	0–No.16	97	78	20	0	..	..	..	..	..	1.95	108
37		0–No. 8	98	80	28	11	0	..	..	..	..	2.17	111
38		0–No. 4	98	82	35	19	9	2	0	..	..	2.45	113
39	Janesville sand and Elgin pebbles...	0–⅜ in.	99	90	63	54	49	45	0	..	..	4.00	123
40		0–¾ in.	99	93	75	69	64	61	39	0	..	5.00	128
1–28, 41		0–1½in.	99	95	80	76	74	71	52	18	0	5.65	130
29		0–1½in.	98	84	43	30	20	14	8	3	0	3.00	118
30		0–1½in.	99	88	57	46	41	35	25	9	0	4.00	127
31		0–1½in.	99	90	64	56	51	46	33	11	0	4.50	131
32		0–1½in.	99	92	71	65	61	57	41	14	0	5.00	132
33		0–1½in.	99	93	75	70	66	62	45	15	0	5.25	133
34		0–1½in.	100	96	86	83	80	78	58	19	0	6.00	127
35		0–1½in.	100	97	89	86	85	84	63	21	0	6.25	124
42–53	Janesville sand and Elgin pebbles.......	0–1½in.	99	95	80	76	74	71	52	18	0	5.65	130
54	Janesville sand and crushed slag........	0–1½in.	99	95	80	76	74	71	52	18	0	5.65	118
55	Janesville sand and crushed limestone...	0–1½in.	99	95	80	76	74	71	52	18	0	5.65	129
56	Janesville sand and crushed granite.....	0–1½in.	99	95	80	76	74	71	52	18	0	5.65	121
57	Washed Elgin sand and Elgin pebbles...	0–1½in.	99	95	86	80	72	67	49	17	0	5.65	127
58	Unwashed Elgin sand and Elgin pebbles...	0–1½in.	100	98	91	86	81	73	50	18	0	5.97	130
59–64	Janesville sand and Elgin pebbles.......	0–1½in.	99	95	80	75	74	71	52	18	0	5.65	130

* Sum of per cents in sieve analysis, divided by 100.

Twin beams were tested in different positions, at different ages, or after storage under different conditions, etc.

The wood forms for the concrete beams were set on a sheet of building paper to prevent bonding with the concrete floor. The forms were thoroughly painted before using and were kept oiled in order to prevent absorption of water. The concrete was placed in two layers, each layer being puddled with a ⅝-in. steel bar pointed at the lower end. After the forms were filled, the concrete was leveled off with a trowel and finished with a wood float. The beams remained in the form over night, being covered with damp burlap sacks. After removal of the forms the beams were stacked compactly on end and kept covered with damp burlap sacks until 28 days old. Beams for test at 3 months and 1 year were placed in the moist room at the age of 28 days. In one group of tests the specimens were stored in a damp condition for different periods, followed by periods of dry air storage.

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[11] Time of Set of Concrete, by Watson Davis; Proc. Am. Soc. Testing Mat., 1921, p. 995.

The concrete cylinders were molded from the same batch from which beams were made. Metal forms made of cold-drawn steel tubing were used.

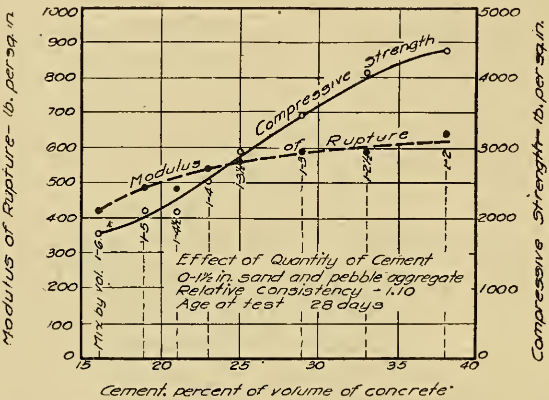

FIG. 3.—EFFECT OF QUANTITY OF CEMENT ON TRANSVERSE AND COMPRESSIVE STRENGTH OF CONCRETE.

Data from Table III.

Plain concrete beams 10 in. wide, 7 in. deep, loaded at ⅓ points of 36-in. span: compression tests of 6 by 12-in. cylinders.

Each point is based on 20 tests made on 10 different days.

Beams stored under damp burlap; cylinders in moist room.

Each form stood on a machined cast-iron base plate. The concrete was puddled in the forms in 3 layers, using a ⅝-in. round bar pointed at the lower end. The forms remained in place over night. After the removal of the forms the specimens were stored in a moist room with the humidity near the saturation point and a temperature of about 65 to 70°F. In making the concrete specimens we followed the methods of the American Society for Testing Materials.[12]

The five or more specimens included in averaging the results for a given set of tests were made on different days. One complete round of specimens was made before beginning the second round. This served to distribute accidental variations of materials, manipulation or test conditions. Any value in the series can be compared directly with any other.

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[12] Tentative Methods for Making Compression Tests of Concrete; Proc. A. S. T. M., 1921, p. 579.

METHODS OF TESTING.

Both beam and cylinder tests were made in a 200,000-lb. Olsen universal testing machine. The beams were loaded at the one-third points of a 36-in. span. They were supported and loaded in such a manner as to prevent restraint. One end rested on a roller and the other on a spherical bearing block. The load was transmitted to the beam through leather cushions which prevented concentration of the load due to minor irregularities in the surfaces. The method of loading is shown in Fig. 1.

Both beams and cylinders were tested in a damp condition immediately after removal from storage, except in the case of the group in which combined moist and dry storage was used.

TEST DATA.

Complete data of the tests are given in Tables III to VIII and in Fig. 3 to 11. Only averages are reported; in most instances the values are the average of 5 tests made on different days.

DISCUSSION OF TESTS.

The flexural strength of concrete beams is measured by the modulus of rupture. The modulus of rupture may be defined as the tensile stress developed by beam action, assuming that the stress and deformation are directly proprotional to the distance from the neutral axis of the beam. The modulus of rupture of a rectangular beam freely supported at the ends and loaded at the one-third points of the span is determined by the following relation:

$$R = \frac{WL}{bd^2} \quad \dots\dots\dots\dots\dots\dots \quad (1)$$

Where R = Modulus of rupture of the material,
W = Total load applied at one-third points,
L = Span length,
b = Breadth of beam,
d = Depth of beam.

In case of most of the beams used in this investigation (width 10 in., depth 7 in., span 36 in., load applied at one-third points) the modulus of rupture is given by the relation:

$$R = 0.073W \quad \dots\dots\dots\dots\dots\dots \quad (2)$$

Where R is expressed in pounds per square inch and W in pounds.

In the reinforced-concrete beams (Table VIII) the modulus of rupture was calculated in the same way, that is, the presence of steel was disregarded.

It will be noted that tests on both beams and cylinders of 1 : 4 mix, aggregate graded 0 to 1½ in., relative consistency 1.10, stored in the moist room and tested in damp condition at age of 28 days appear in practically all tables and figures. This was considered a *typical* mix for concrete roads. These conditions gave a modulus of rupture of 550 lb. per sq. in. and a compressive strength of 2580 lb. per sq. in. These values are based on 25 beam tests and 115 cylinder tests, made throughout the series. It is interesting to note how this group of tests falls into each of the classifications into which the data have been divided.

With few exceptions the beams failed by nearly vertical cracks between the load points.

Effect of Position of Beam During Test. In general the beams were loaded as molded; that is, with the bottom of the beam in tension. It seemed important to determine whether the position of the beam exerted any influence on its flexural strength. In one group of tests the beams were tested with the bottom in tension and with bottom in compression. Results of these tests are given in Table III. In the first group the mixture varied from 1: 6 to 1: 2; in the second group the mixture was 1: 4 by volume, but the consistency varied from relative consistency 0.90 to 1.50. Aggregates in all cases consisted of a mixture of sand and pebbles graded up to 1½ in. Both groups of tests show that there is little or no difference in the flexural strength of the beam due to the position of the load; the only exception to this statement is found in the wetter mixtures which gave a higher modulus of rupture with the *top* in tension.

Effect of Quantity of Cement. The tests in Table III also give information on the effect of quantity of cement on the flexural strength of concrete. The mixture varied from 1: 6 to 1: 2. Each point on the modulus of rupture curve in Fig. 3 is based on the average of tests of 20 beams (10 with top as molded in tension and 10 with bottom in tension). Increasing the quantity of cement increased both the flexural and the compressive strength of the concrete; however, the modulus of rupture was less affected by the quantity of cement than the compressive strength. The modulus of rupture was relatively lower for the rich mixtures than for the lean, as compared with the compressive strength of 6 by 12-in. cylinders from the same concrete. The modulus of rupture for the richer mixes (1: 2 and 1: 2½) was about 14% of the compressive strength; for the leaner mixtures (1: 6 and 1: 5) about 23% of the compressive strength; for a 1: 4 mix (about the same as the 1: 1½: 3 or 1: 2: 3 mixes usually specified for concrete roads) the modulus of rupture was about 21% of the compressive strength of 6 x 12-in. cylinders. For the usual range of mixtures (1: 4 to 1: 5) the ratio was about 23%.

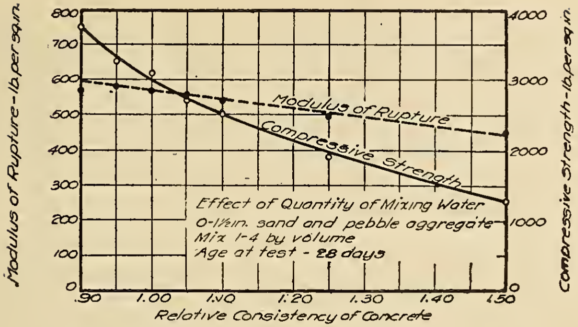

FIG. 4.—EFFECT OF QUANTITY OF MIXING WATER.

Data from Table III.
Each point is based on 20 tests of each kind, made on 10 different days.

TABLE III.—EFFECT OF QUANTITY OF CEMENT AND MIXING WATER.

Aggregate: sand from Janesville, Wis., and pebbles from Elgin, Ill.; graded up to 1½ in. Fineness modulus, 5.65.

Age at test, 28 days.

Specimens tested damp.

Each value for modulus of rupture is the average of 10 tests, and for compressive strength 20 tests, made on 10 different days.

Ref. No.	Mix by Volume.	Cement, per cent of Volume of Concrete.	Relative Consistency.	Water-Ratio of Concrete.	Modulus of Rupture of Beams, lb. per sq. in.			Compressive Strength of 6 x 12-in. Cylinders, lb. per sq. in.	Modulus of Rupture, per cent of Compression.
					Bottom*	Top.*	Aver.		
EFFECT OF QUANTITY OF CEMENT. (See Fig. 3.)									
1, 2......	1 : 6	16.4	1.10	1.03	430	420	425	1820	23.4
3, 4......	1 : 5	19.0	1.10	0.92	500	490	495	2140	23.1
5, 6......	1 : 4½	20.7	1.10	0.87	480	510	495	2130	23.2
7, 8......	1 : 4	23.0	1.10	0.82	560	540	550†	2580†	21.3
9, 10.....	1 : 3½	25.4	1.10	0.76	590	540	565	2980	19.0
11, 12....	1 : 3	28.7	1.10	0.71	600	590	595	3480	17.1
13, 14....	1 : 2½	33.0	1.10	0.64	590	590	590	4110	14.3
15, 16....	1 : 2	38.7	1.10	0.59	660	620	640	4390	14.6
				Average	550	540	545	2950	19.5
EFFECT OF QUANTITY OF MIXING WATER. (See Fig. 4.)									
17, 18....	1 : 4	23.8	0.90	0.68	590	560	575	3760	15.3
19, 20....	1 : 4	23.5	0.95	0.72	580	600	590	3280	18.0
21, 22....	1 : 4	23.4	1.00	0.75	580	560	570	3100	18.4
23, 24....	1 : 4	23.2	1.05	0.78	570	550	560	2720	20.6
7, 8......	1 : 4	23.0	1.10	0.82	560	540	550†	2580†	21.3
25, 26....	1 : 4	22.5	1.25	0.92	460	540	500	1920	26.0
27, 28....	1 : 4	22.3	1.50	1.08	400	500	450	1300	34.6
				Average	535	550	540	2660	22.0

* Part of concrete beam (as molded) which was exposed to tensile stress during loading.

† Average of 25 beam tests and 115 cylinder tests; the same values are used in Tables IV to VIII.

Effect of Quantity of Mixing Water is shown in Table III and Fig. 4. Tests were made on 1 : 4 mix at age of 28 days. The quantity of mixing water is expressed in terms of relative consistency and the water-ratio of the concrete. Consistency 1.00 is such as to give a flow of 180. A relative consistency of 0.90 is about as dry a mix as can be considered plastic with this method of molding specimens; a consistency of 1.50 is quite wet and shows considerable free water on top of the concrete. Both the modulus of rupture and compressive strength were reduced by increasing the quantity of mixing water. However, the modulus of rupture was less affected than the compressive strength. For the drier mixtures the modulus of rupture was 15% of the compressive strength; for the wetter mixtures about 35%. While the flexural strength of the wetter mixtures is *relatively* higher than for the drier ones, it should be noted that the modulus of rupture of consistency 1.50 is 450, while the values for consistencies 0.95 to 1.10 averaged about 570 lb. per sq. in.—an increase of 26%. The latter consistencies represent those which should be aimed at

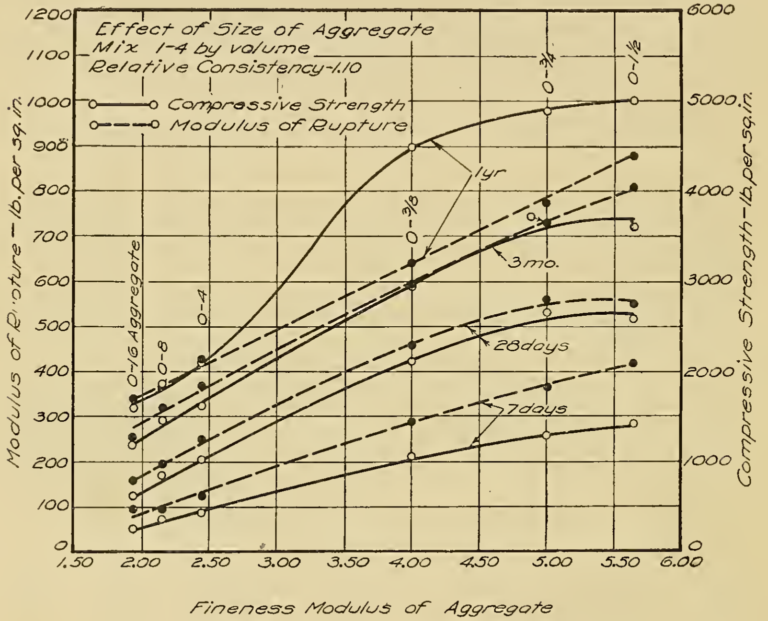

FIG. 5.—EFFECT OF SIZE OF AGGREGATE.

Data from Table IV.
Janesville sand and Elgin pebbles; see Table II for sieve analysis.
Each point is based on 5 tests of each kind made on different days.

in concrete road construction. These tests confirm other tests made in this Laboratory and elsewhere in showing the importance of restricting the quantity of mixing water used in concrete.

Effect of Size of Aggregate is shown in Table IV and Fig. 5. Tests were made on 1 : 4 mixtures, using aggregates varying in size from 16-mesh sand to a concrete aggregate graded up to 1½ in. The "effective grading" of the aggregate is measured by the fineness modulus. Tests were made at ages of 7 days, 28 days, 3 months and 1 year. With the same quantity of cement, the modulus of rupture and the compressive strength of the concrete increased rapidly as the size of aggregate was increased, up to a point where the aggregate became too coarse for the quantity of cement. The rate of increase of strength of concrete was nearly pro-

TABLE IV.—EFFECT OF SIZE AND GRADING OF AGGREGATES.

Aggregates: sand from Janesville, Wis., and pebbles from Elgin, Ill. Aggregates of *different size* were obtained by separating sand and pebbles into various sizes and recombining as shown by sieve analyses in Table II and Fig. 1. *Different gradings* of aggregates were produced by mixing sand (0 to No. 4) and pebbles (No. 4 to 1½ in.) in different proportions.

Mix, 1 : 4 by volume. Relative consistency, 1.10.

Specimens tested damp.

Each value is the average of 5 tests made on different days.

Ref. No.	Aggregate.		Water-Ratio of Concrete.	Modulus of Rupture of Beams, lb. per sq. in.				Compressive Strength 6 x 12-in. Cylinder, lb. per sq. in.				Modulus of Rupture, per cent of Compression.			
	Size.	Fineness Modulus.		7 d.	28 d.	3 m.	1 y.	7 d.	28 d.	3 m.	1 y.	7 d.	28 d.	3 m.	1 y.
EFFECT OF SIZE OF AGGREGATE. (See Fig. 5.)															
36	0–16	1.95	1.29	95	160	255	340	270	620	1190	1600	35.2	25.8	21.5	21.2
37	0–8	2.17	1.25	95	195	320	370	360	850	1470	1860	26.4	23.0	21.8	19.9
38	0–4	2.45	1.20	125	250	370	425	430	1010	1620	2100	29.4	24.8	22.8	20.2
39	0–⅜	4.00	0.98	290	455	595	640	1040	2110	2930	4490	27.9	21.6	20.3	14.3
40	0–¾	5.00	0.87	365	560	730	775	1290	2650	3650	4890	28.3	21.2	20.0	15.9
41	0–1½	5.65	0.82	420	550*	810	880	1410	2580	3590	5000	29.8	21.3	22.6	17.6
			Aver.	230	360	510	570	800	1640	2410	3320	29.5	22.9	21.5	18.2
EFFECT OF GRADING OF AGGREGATE. (See Fig. 6.)															
29	0–1½	3.00	1.11	165	255	410	450	620	1290	1640	2330	26.6	19.8	25.0	19.3
30	0–1½	4.00	0.98	230	390	505	570	950	2000	2550	3230	24.2	19.5	19.8	17.7
31	0–1½	4.50	0.93	285	485	610	645	1090	2190	2750	3830	26.2	22.2	22.2	16.9
32	0–1½	5.00	0.87	325	505	660	710	1160	2410	3580	4510	28.0	21.0	18.4	15.8
33	0–1½	5.25	0.85	365	555	735	820	1320	2940	3810	5340	27.7	18.9	19.3	15.4
41	0–1½	5.65	0.82	420	550*	810	880	1410	2580*	3590	5000	29.8	21.3	22.6	17.6
34	0–1½	6.00	0.78	405	600	735	825	1300	2250	3310	4400	31.2	26.7	22.2	18.8
35	0–1½	6.25	0.77	385	590	730	865	1140	1990	2840	4080	33.8	29.6	25.7	21.2
			Aver.	320	490	650	720	1120	2210	3010	4090	28.4	22.4	21.9	17.8

* Average of 25 beam tests and 115 cylinder tests; same values used in Tables III, V, VI, VII and VIII

portional to the change in fineness modulus of the aggregate. The modulus of rupture at 28 days varied from 160 to 560 lb. per sq. in.; and the compressive strength from 620 to 2650 lb. per sq. in. Except for the 16-mesh sand the ratio between the modulus of rupture and the compressive strength was nearly constant for a given age—21 to 23% at 28 days. In other words the transverse and the compressive strengths are affected in a similar manner by differences in the size of aggregate, as shown by the practically parallel curves for a given age in Fig. 5.

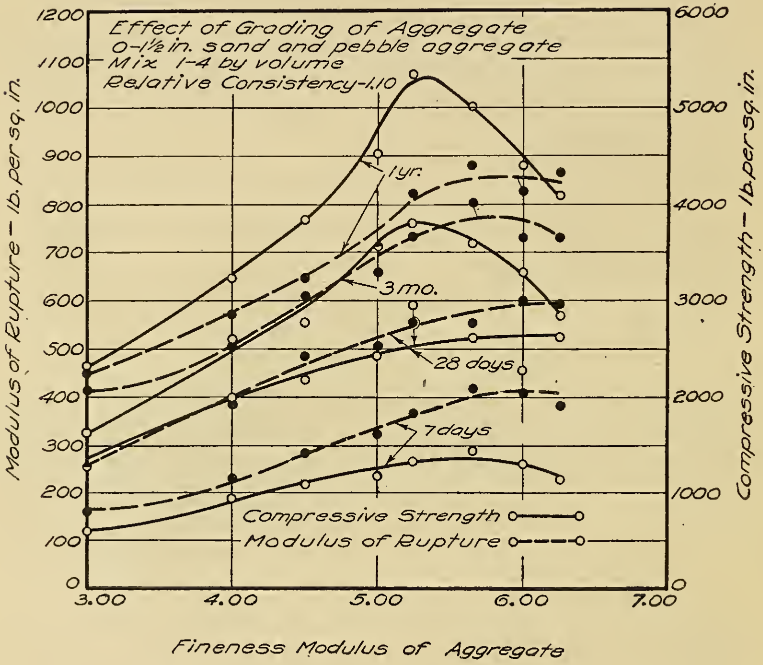

FIG. 6.—EFFECT OF GRADING OF AGGREGATE.

Data from Table IV.

Aggregate gradings were obtained by mixing Janesville sand (0 to No. 4) and Elgin pebbles (4 to 1½-in.) in different proportions; sieve analysis given in Table II.

Each point is based on 5 tests of each kind made on different days.

Effect of Grading of Aggregate is shown in Table IV and Fig. 6. Tests were made on a 1 : 4 mix at ages of 7 days, 28 days, 3 months and 1 year. The aggregates of different grading used in these tests were produced by mixing different proportions of sand (0 to No. 4) and pebbles (No. 4 to

1½ in.). The finest grading consisted of 86% sand and 14% pebbles; the coarsest grading consisted of 14% sand and 86% pebbles, as shown by the sieve analyses in Table I. With the same quantity of cement, the modulus of rupture and the compressive strength increased rapidly as the grading of the aggregate was made coarser (by increasing the relative proportion of pebbles) until a grading was reached which was too coarse for the quantity of cement. The flexural and compressive strength were affected in essentially the same way by changes in grading of the aggregate. The rate of increase of strength of the concrete was nearly proportional to the increase in fineness modulus of the aggregate. If the values in Fig. 5 and 6 are platted on the same diagram it will be found that they are practically coincident, although the relation between the strength and fineness modulus of the aggregate is based on aggregate gradings made up in entirely different ways.

Effect of Curing Condition of Concrete; mix 1:4 tested at the age of 28 days. The results are given in Table V and Fig. 7. The curing conditions ranged from 28 days under damp burlap (moist room for cylinders) to 28 days in air of the Laboratory. The flexural strength and the compressive strength were affected in the same way, as shown by the similarity of the curves in Fig. 7. The modulus of rupture of beams cured in dry air for the entire period of 28 days was 370 lb. per sq. in.; cured under damp burlap for 28 days was 550 lb. per sq. in.; an increase of 48%. Curing 7 days under damp burlap and 21 days in dry air gave 470 lb. per sq. in.; an increase of 27% over the dry air storage for 28 days. It will be recalled that the specimens which were cured under the most unfavorable conditions were prevented from drying out for the first 24 hr.; had they been allowed to dry out from the beginning the modulus of rupture and compressive strength would have been still lower. These tests show the great advantage of damp curing on both the flexural and compressive strengths and confirm the results of earlier tests[13] made in this Laboratory and elsewhere.[14]

Effect of Depth of Beam, 1:4 mix, tested at 28 days. The depth of beam varied in this group from 4 to 10 in. Data are given in Table VI and Fig. 8. The curves show that the modulus of rupture was only slightly affected by the depth of the beam within the range of these tests. There is an indication of somewhat lower values for the deeper beams. The 4-in. beams gave a modulus of rupture of 545 lb. per sq. in.; the 8 and 10-in. beams averaged 525 lb. per sq. in. Flexural tests of other materials have shown that wide variations in ratio of depth of beam to span produce important variations in modulus of rupture; in general, the deeper beams give the higher values.

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[13] Effect of Curing Condition on the Wear and Strength of Concrete, by Duff A. Abrams; Bull. 2, Struc. Mat. Research Lab.

[14] Relation between Methods of Curing Standard Concrete Test Specimens and Their Compressive Strength at 28 days, by H. W. Green; Proc. Am. Soc. Testing Mat., 1919, Part II, p. 607.

Effect of Kind of Aggregate. Tests were made on 1: 4 mix at age of 7 days, 28 days, 3 months and 1 year, using the following aggregates:

(1) Janesville, Wis., sand
  (a) Elgin pebbles (limestone origin)
  (b) Crushed blast furnace slag
  (c) Crushed limestone
  (d) Crushed granite (Wisconsin)

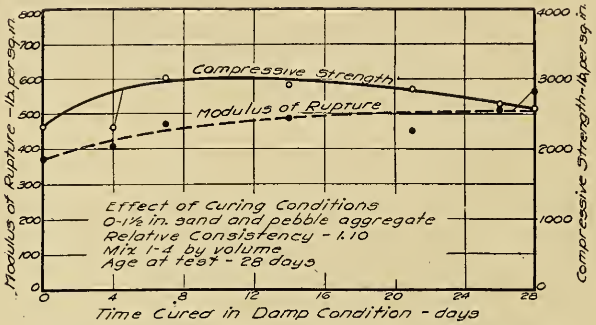

FIG. 7.—EFFECT OF CURING CONDITION OF CONCRETE.

Data from Table V.

Each point is based on 5 tests of each kind made on different days.

Beams cured under damp burlap, cylinders in moist room, for periods shown by abscissae.

TABLE V.—EFFECT OF CURING CONDITION OF CONCRETE.

Mix, 1 : 4 by volume. Relative consistency of concrete, 1.10; water-ratio, 0.82.
Age at test, 28 days.
Aggregate: sand from Janesville, Wis., and pebbles from Elgin, Ill.; graded up to 1½ in.
Each value is the average of 5 tests made on different days.
Data platted in Fig. 7.

Ref. No.	Storage.		Modulus of Rupture of Beams, lb. per sq. in.	Compressive Strength of 6 x 12-in. Cylinders, lb. per sq. in.	Modulus of Rupture, per cent of Compression.
	Damp Burlap.	Dry Air.			
7, 8	28 d.	...	550*	2580*	21.3
42	26 d.	2 d.	510	2630	19.4
43	21 d.	7 d.	450	2850	15.7
44	14 d.	14 d.	485	2920	16.6
45	7 d.	21 d.	470	3020	15.6
46	4 d.	24 d.	410	2330	17.6
47	0 d.	28 d.	370	2340	15.8
			Average 465	2670	17.5

* Average of 25 beam tests and 115 cylinder tests; same values used in Tables III, IV, VI, VII, and VIII.

7 and 28-day tests only were made on the following aggregates:

(2) Elgin, Ill., sand (limestone origin) washed
    (a) Elgin pebbles.
(3) Elgin sand—unwashed
    (a) Elgin pebbles.

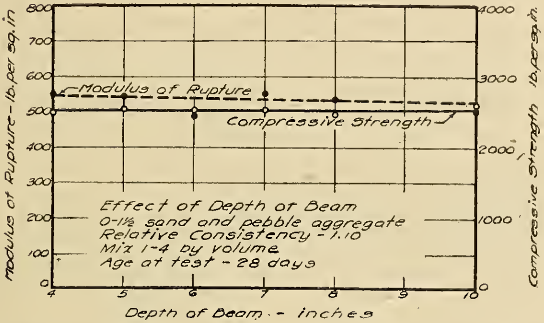

FIG. 8.—EFFECT OF DEPTH OF BEAM ON THE TRANSVERSE STRENGTH OF CONCRETE.

Data from Table VI.

Plain concrete beams 4 to 10 in. deep, 10 in. wide, loaded at ⅓ points of a 36-in. span.

Each point is based on 5 tests of each kind made on different days.

TABLE VI.—EFFECT OF DEPTH OF BEAM ON THE FLEXURAL STRENGTH OF CONCRETE.

Mix, 1 : 4 by volume. Relative consistency of concrete, 1.10; water-ratio, 0.82.
Aggregate: sand from Janesville, Wis., and pebbles from Elgin, Ill.; graded up to 1½ in.
Age at test, 28 days.
Specimens tested damp.
Each value is the average of 5 tests made on different days.
Data platted in Fig. 8.

Ref. No.	Depth of Beam, inches.	Modulus of Rupture of Beams, lb. per sq. in.	Compressive Strength of 6 x 12-in. Cylinders, lb. per sq. in.	Modulus of Rupture, per cent of Compression.
48	4	545	....	21.2
50	5	540	....	20.9
52	6	490	2580*	19.0
53	7	550*	....	21.3
51	8	535	....	20.8
49	10	510	....	19.8
		Average 530	....	20.6

* Average of 25 beam tests and 115 cylinder tests; same values used in Tables III, IV, V, VII and VIII.

The coarse aggregate (and consequently the total aggregate in each batch) were similarly graded. The sieve analyses are given in Table II and Fig. 2. There was no great discrepancy between the flexural and compressive strengths given by the pebbles, slag and limestone; the granite gave somewhat lower values both in flexure and compression. The reason for this probably lies in the low absorption of the granite. The absorption of water by the other aggregate resulted in a lowering of the water-ratio of the batch, although the concrete was mixed to the same consistency or workability at the beginning. The washed Elgin sand (silt 0.5%) gave values for modulus of rupture a little lower than unwashed (silt 2.0%), but the compressive strength was slightly higher, indicating that there is little difference in strength of concrete due to slight differences in silt content. The objection to the use of sands of high silt content is not that the strength of concrete of *proper* consistency is materially reduced, but that with wet mixes frequently used in concrete road construction the silt is floated to the surface where it produces a weak and friable concrete.

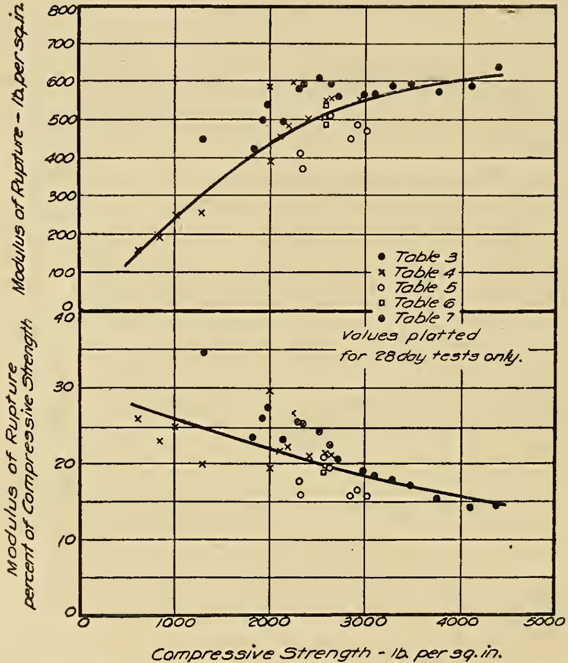

FIG. 9.—RELATION BETWEEN THE TRANSVERSE AND COMPRESSIVE STRENGTH OF CONCRETE.

Data from Tables III to VII.

Plain concrete beams 10 in. wide, 7 in. deep, loaded at ⅓ points of a 36-in. span; compression tests of 6 by 12-in. cylinders.

Effect of Age of Concrete. In Tables IV and VII, both flexural and compression tests are reported on 1:4 mixes at ages of 7 days, 28 days,

TABLE VII.—EFFECT OF KIND OF AGGREGATE.

Mix, 1 : 4 by volume.

Relative consistency, 1.10; water-ratio, 0.82.

Aggregate: sand, 0 to No. 4; and coarse aggregate, No. 4 to 1½ in.; all of same grading.

Specimens tested damp.

Age at test, 28 days.

Each value is the average of 5 tests made on different days.

Ref. No.	Kind of Aggregate.		Modulus of Rupture of Beams, lb. per sq. in.				Compressive Strength of 6 x 12-in Cylinders, lb. per sq. in.				Modulus of Rupture, per cent of Compression.			
	Sand.	Coarse.	7 d.	28 d.	3 m.	1 y.	7 d.	28 d.	3 m.	1 y.	7 d.	28 d.	3 m.	1 y.
7,8	Janesville.....	Elgin pebbles	420	550*	810	880	1410	2580*	3590	5000	29.8	21.5	22.6	17.6
54	Janesville.....	Slag........	450	585	765	760	1240	2300	3150	4530	36.3	25.4	24.3	16.8
55	Janesville.....	Limestone ..	440	595	790	830	1320	2350	3280	3970	33.4	25.3	24.1	20.9
56	Janesville.....	Granite.....	375	540	665	725	1010	1980	2960	3760	37.1	27.3	22.4	19.3
57	Elgin washed..	Elgin pebbles	405	595	...	...	1490	2640	....	....	27.2	22.5	....	....
58	Elgin unwashed	Elgin pebbles	425	610	...	...	1340	2520	....	....	31.7	24.2	....	....
		Average	420	580	760	800	1300	2390	3240	4320	32.6	24.3	23.4	18.6

* Averge of 25 beam tests and 115 cylinder tests; same values used in Tables III, IV, V, VI and VIII.

TABLE VIII.—EFFECT OF NUMBER OF STEEL REINFORCING BARS.

Beams: Depth, 7 in.; width, 10 in.; span, 36 in.

Longitudinal reinforcing bars were spaced equally across the width and 1 in. from the bottom of the beam. The total depth of beams varied from 6.9 to 7.1 in.

Mix, 1 : 4 by volume. Aggregate: sand and pebbles graded 0 to 1½ in.

Relative consistency of concrete, 1.10; water-ratio, 0.82.

Age at test, 28 days.

Specimens tested damp.

Each value is the average of 5 tests made on different days.

Compressive strength of 6 by 12-in. concrete cylinders (average of 115 tests) was 2580 lb. per sq. in.

Ref. No.	Number of ⅜-in. Round Steel Bars.	Percentage of Steel.	Total Load on Beam, pounds.	Computed Stresses, lb. per sq. in.					Type of Beam Failure.
				Tension in Steel.	Compression in Top Fibre.	Bond.	Shearing Unit Stress.	Modulus of Rupture.	
7,8	0	None	7,870	......	....	...	70	550†	Tension
59	1	0.18	7,850	73,000*	1470*	570	70	560*	Tension
60	2	0.36	10,390	49,400*	1460*	380	90	730*	Tension
61	3	0.54	15,010	49,100*	1850*	380	140	1085*	Tension
62	4	0.72	19,640	48,900*	2190*	380	180	1435*	Diagonal shear
63	6	1.09	25,800	43,800*	2500*	340	240	1885*	Diagonal shear
64	8	1.46	28,060	36,800*	2530*	290	270	1990*	Diagonal shear

* Tension in concrete not considered in computing steel stress. Steel not considered as taking stress in computing modulus of rupture.

† Average of 25 beam tests; same values used in Tables III to VII.

3 months and 1 year. The values from these tests are platted in Fig. 11. These tests show that both the flexural and compressive strength are proportional to the logarithm of the age of the concrete. The flexural strength increased less rapidly than compression, as shown by the average ratio of flexure to compressive strength of 30% at 7 days, 23% at 28 days, 22% at 3 months and 18% at 1 year. Other studies of the effect of age on the compressive strength of concrete[15] have shown that this logarithmic relation holds indefinitely so long as the concrete does not dry out.

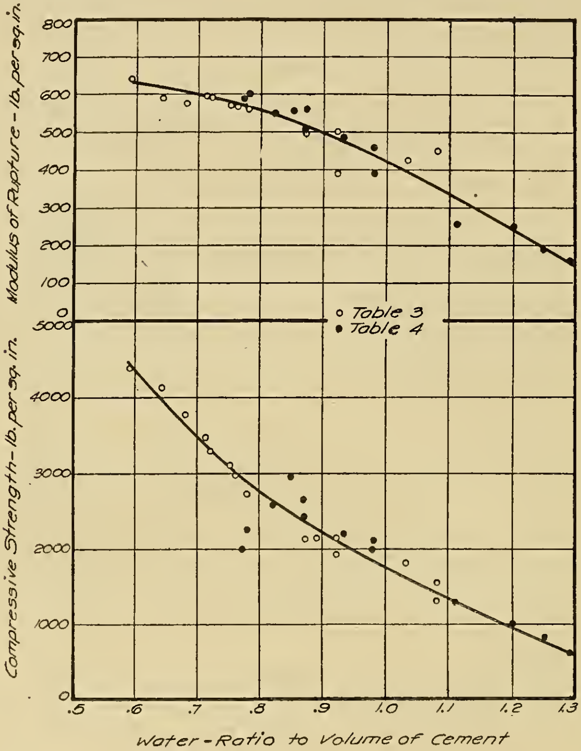

FIG. 10.—RELATION BETWEEN STRENGTH AND WATER-RATIO OF CONCRETE.

Data from Tables III and IV.

Plain concrete beams 10 in. wide, 7 in. deep, loaded at ⅓ points of a 36-in. span; compression tests of 6 by 12-in. cylinders.

Relation between Modulus of Rupture and Compressive Strength. Many of the tables show the modulus of rupture in terms of the com-

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[15] Effect of Age on the Strength of Concrete, by Duff A. Abrams, Proc. Am. Soc. Testing Mat., 1918, Part II; see also Concrete, July, 1921, p. 14.

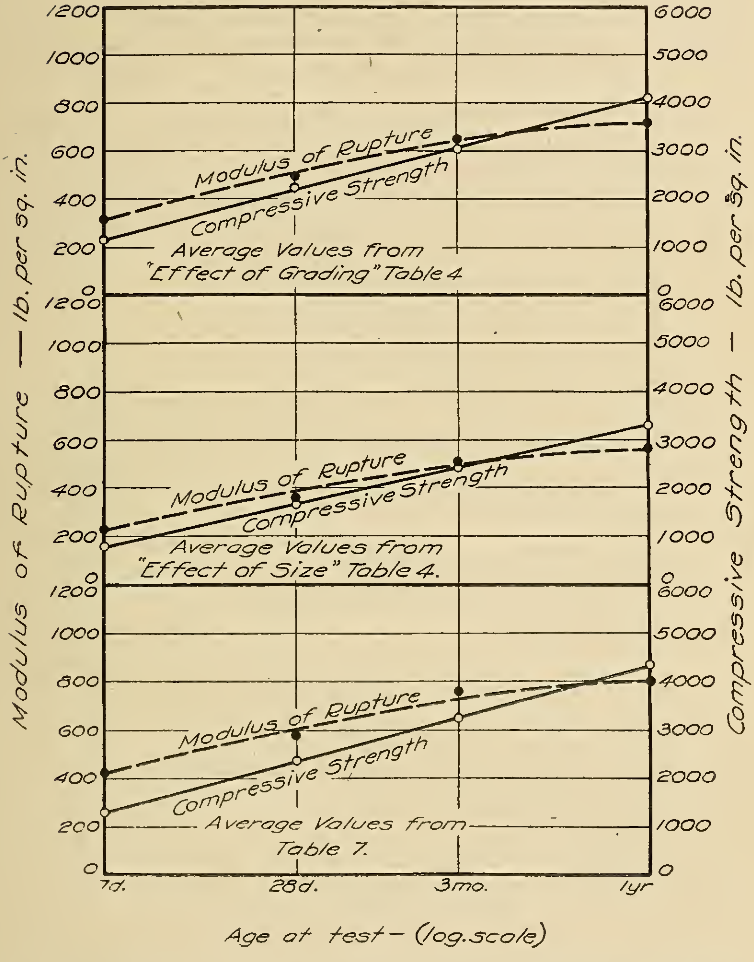

FIG. 11.—EFFECT OF AGE ON THE TRANSVERSE STRENGTH OF CONCRETE.

Data from Tables IV and VII.

Each point is the average of several different gradings, or kinds of aggregate.

Beams cured under damp burlap; cylinders in moist room.

pressive strength of the concrete. The values for 28-day tests from Tables III to VII inclusive are platted in Fig. 10. The upper diagram gives the actual values and shows a fairly consistent relation between the modulus of rupture and the compressive strength regardless of the mixture, con-

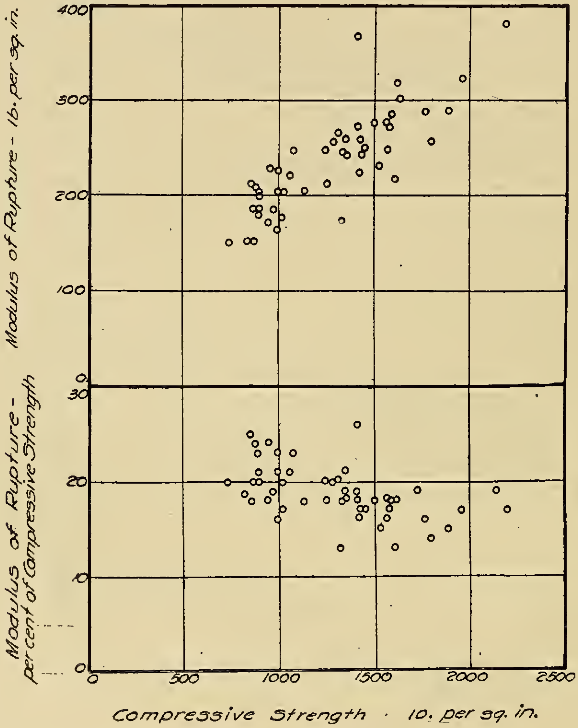

FIG. 12.—RELATION BETWEEN TRANSVERSE AND COMPRESSIVE STRENGTH OF CONCRETE.

BY FULLER AND THOMPSON.

From "Laws of Proportioning Concrete." Trans. Am. Soc. Civil Eng., v. 59, p. 67 (1907).

sistency, size and grading of aggregate, etc. This relation may be expressed by the equation:

$$R = 0.29S - 0.000032S^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where R = modulus of rupture of plain beams
S = compressive strength of 6 by 12-in. cylinder.

This is the equation of the upper curve in Fig. 5 when stresses are given in pounds per square inch. The modulus of rupture is relatively higher for the weak concrete; at 28 days it is about 26% of the compressive strength for 1000-lb. per sq. in. concrete; 22% for 2000-lb.; 19% for 3000-lb. and 15% for 4000-lb. concrete.

Relation between Strength and Water-Ratio of Concrete. The water-ratio is defined as the ratio of volume of water to volume of cement in the batch. A water-ratio of 0.75 to 1.00 covers the usual range of mixtures. In platting Fig. 9 only 28-day tests from Tables III and IV were used. It will be noted that the water-ratio was varied, due to the following causes:

(1) Quantity of cement,
(2) Consistency of concrete,
(3) Size of aggregate,
(4) Grading of aggregate.

In spite of the wide differences in the *cause* of variation of water-ratio, it is found that the relation between strength and water-ratio can be expressed by a single curve for each form of test. The lower curve shows the usual relation which has been found between the compressive strength and water-ratio.[16] The upper curve shows the relation between water-ratio and modulus of rupture of concrete to be of an entirely different form from that found for compression. The compressive strength drops off rapidly at first, then less rapidly with an increase in the quantity of mixing water. The modulus of rupture drops off slowly at first, then more rapidly. The comparison between these relations may be best seen by a specific example. Fig. 9 shows that by increasing the water-ratio from 0.80 to 1.20 (an increase of 50% in mixing water) the compressive strength is reduced from 2750 to about 950 lb. per sq. in.; and the modulus of rupture from 550 to 240 lb. per sq. in. In other words, the wetter concrete gives a compressive strength of about ⅓ and a modulus of rupture of about ½ that produced by the concrete in which care was taken to restrict the mixing water to a reasonable quantity.

Effect of Number of Steel Reinforcing Bars. In one group of tests, beams of standard size were reinforced with 1 to 8 ⅜-in. round steel bars, placed with centers 6 in. below the top of the beam. Table VIII gives total loads, the computed tensile stress in the steel, compressive stress in the top fiber of beams, the bond stress between the concrete and steel, shearing unit stress and modulus of rupture. In calculating the tension in the steel, the stresses carried by the concrete was not considered; in computing the modulus of rupture the stress carried by the steel was not considered. The tests show that no effect is produced by one reinforcing bar. Upon failure of the concrete the stress is transmitted to the reinforcing bar. The stress carried by one bar when stressed to the ultimate simply counterbalances the failure of the concrete in tension. With 4 to 8 bars, failure was produced by diagonal tension which did not develop the tensile strength of the steel.

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[16] Design of Concrete Mixtures, by Duff A. Abrams; Bull. 1, Structural Materials Research Lab., 1918.

Uniformity of Tests. The tables and diagrams show unusual uniformity in the tests in this series. In only a few instances are the points off the smooth curves which represent the relations between strength and other factors. The strength of concrete depends on so many factors; it is inevitable that some discrepancies should appear. The making of five or more test pieces on different days seems to eliminate most accidental variations. The methods followed in making these tests are recommended to others who may be engaged in concrete testing.

EARLIER FLEXURAL TESTS OF CONCRETE.

Fig. 12 gives the results of flexural and compression tests of concrete made by Fuller and Thompson, and published in their paper on Laws of Proportioning Concrete.[17] These tests cover a narrow range of strengths, but in general, they show about the same general relation as that found in Fig. 10.

Fig. 13 gives results of tests made by R. Feret[18] on mortar beams 4 by 4 by 16 cm. The relation between modulus of rupture and compressive strength is somewhat similar to that given in Fig. 10.

Flexural tests of mortars were carried out by the U. S. Geological Survey at St. Louis, 1905-08.[19]

FLEXURAL STRENGTH OF OTHER MATERIALS.

The following notes on the flexural and compressive strength of other materials than concrete may be of interest:

Marble. Tests made by the Bureau of Standards on 2¼-in. cubes in compression and beams 1½ to 2 in. deep, 3 to 4 in. wide, span 6 to 10 in. show that the modulus of rupture is uniformly about 15% of the compressive strength for marbles ranging in strength from 8000 to 28,000 lb. per sq. in. The tensile strength of marble briquets 1-in. square was about one-half the modulus of rupture. (Tech. Paper 123, Tests of Commercial Marbles, by D. W. Kessler, 1919.)

Slate. Tests made by M. O. Fuller showed modulus to vary from 75 to 100% of compressive strength. Modulus of rupture tests were made on beams 1 in. wide, 26 in. long and ¼, ½, 1 and 1½ in. deep; compression tests were made on 2-in. cubes. (Physical and Electrical Properties of Slate, Bull. Lehigh University, 1921.)

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[17] Trans. Am. Soc. Civil Eng., V. 59, p. 67, 1907.

[18] Etude Experimentale du Ciment Arme, p. 576, 1906.

[19] Portland Cement Mortars and Their Constituent Materials, by Humphrey and Jordan. Bull. 331, U. S. Geol. Survey, 1908.

Building Brick. As a result of an exhaustive series of tests carried out by the Committee on Brick of the American Society for Testing Materials, the following values were found for different classes of building brick:

Class	*Modulus of Rupture*	*Compression*	*Ratio, percent*
A	1482	8493	17.5
B	668	4171	16.0
C	621	2791	22.0
D	469	1762	27.0

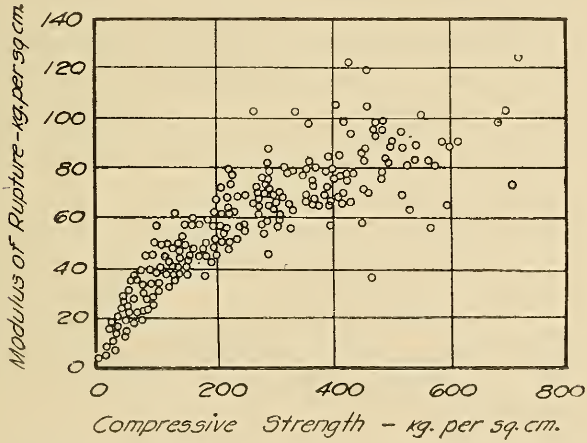

FIG. 13.—RELATION BETWEEN TRANSVERSE AND COMPRESSIVE STRENGTH OF CONCRETE.

BY FERET.

From "Etude Experimentale du Ciment Armé," p. 576 (1906).

Compression tests were made on whole brick, loaded flatwise; modulus of rupture tests on brick loaded flatwise on 7-in. span. (Proc. A. S. T. M., 1915, Part I.)

Gypsum. Tests made by W. A. Slater showed modulus of rupture of gypsum 43 to 52% of compressive strength; 6 x 6 x 18-in. beams and 8 x 16-in. cylinders, 35% water, age at test, 10 days. (Jl. West. Soc. Eng., Sept., 1919, p. 441.)

Cast-iron. Kirkaldy's tests on 469 samples of cast-iron showed an average modulus of rupture of 38,000 and a compressive strength of 121,000 lb. per sq. in. (ratio 31%). Modulus of rupture tests were made on bars 2 x 1 x 36-in. loaded edgewise. (Materials of Construction, by J. B. Johnson, 4th Ed., 1898, p. 447.)

CONCLUSIONS.

The following conclusions may be stated:

The tests give definite results on the flexural and compressive strength of concrete for a wide range of mixtures, consistencies, size and grading of aggregates, curing conditions, kind of aggregate, etc.

The relation between modulus of rupture and compressive strength is not uniform; for the weak mixtures the modulus of rupture is *relatively* higher than for the strong mixtures. An equation is given which expresses this relation. For usual concrete mixtures the modulus of rupture under the conditions of these tests, was about 22% of the compressive strength.

Essentially the same strength was developed by the beams loaded with top in tension and with bottom in tension; in the wetter mixtures the values were somewhat lower for bottom in tension.

Increasing the quantity of cement increased both flexural and compressive strength of the concrete. The modulus of rupture was less affected by variations in the quantity of cement than the compressive strength.

Both modulus of rupture and compressive strength of the concrete were reduced by increasing the quantity of mixing water. The modulus of rupture was less affected by the change in mixing water than the compressive strength.

With the same quantity of cement, the modulus of rupture increased rapidly as the size of the aggregate increased or the grading became coarser (up to the point where the aggregate became too coarse for the quantity of cement). The rate of increase in strength was essentially the same in beams and cylinders.

The flexural and compressive strength of concrete were affected in the same way by variations in curing condition of the concrete. The advantage of curing in damp condition was fully brought out by the tests. Curing in damp condition for 28 days gave a modulus of rupture of 48% higher than 1 day in damp condition and 27 days in dry air.

The modulus of rupture of concrete was only slightly affected by the depth of the beam. The values for beams 8 and 10 in. deep were about 4% higher than for the 4-in. beams.

There was no great discrepancy between the flexural and compressive strength of concrete made from pebbles, crushed slag and crushed limestone. Granite gave somewhat lower values, probably due to low absorption of the aggregate.

Both flexural and compressive strength of concrete cured in a damp condition until tested were proportional to the logarithm of the age.

These tests show the same relation between water-ratio and compressive strength of concrete that has been found in earlier tests. However, the relation between water-ratio and modulus of rupture was of an entirely different form.

The methods of test used in this investigation were found to give satisfactory results as shown by the uniformity of the values in the various groups of tests.

DISCUSSION.

Mr. Slater. W. A. Slater.—I think it may be interesting to call attention to the fact that the tests that were made by the Bureau of Mines under the direction of Mr. Humphrey, and studied and reported on in some detail a year ago at this convention,* confirm the conclusion that about 0.2 per cent is the amount of reinforcement below which the beam is just as strong without as with reinforcement. In other words, the beam with about 0.2 per cent of reinforcement fails with about the same load as though it had no reinforcement at all. Four kinds of concrete were used, the strength of which varied from 2200 to 4200 lb. per sq. in. The reinforcement had a yield point of about 40,000 lb. per sq. in.

Mr. Zesiger. A. W. Zesiger.—In regard to too high a percentage of reinforcement, I would like to ask some more questions. I understand that under Mr. Humphrey some years ago they made tests of about 300 concrete beams reinforced from about 0.4 per cent to over 2 per cent, and the report states there very clearly that all the failures were due to the stress in the steel having reached the elastic limit when the beam failed, not due to the failure of the concrete. In those tests was used cinder concrete, which is not very strong, limestone, granite and gravel concrete. If then the beam fails due to some weakness in the concrete, such as shear or bond, why did it not fail in the same manner under those tests?

Mr. Richart. F. E. Richart.—I would like to offer a comment on the matter of curing which Professor Abrams has mentioned here in showing that specimens which were stored damp for perhaps 14 days and then allowed to dry out for 14 days gave greater strengths than those that were cured for 28 days in a damp condition and then tested damp. Two effects are evidently involved here; the difference in the rate of gain in strength during the damp and dry storage periods, and the difference in the strength of the same concrete when moist and when dry. The latter condition exists in stone, brick and other materials; it is of considerable importance in the case of concrete road slabs where moisture is always present in the subgrade and to some extent on the upper surface of the slab. The moisture content of the concrete seems to have a definite effect on the strength.

A number of tests on the effect of storage conditions and of moisture content on the strength of concrete have been made at the University of Illinois. In one series, made in 1915 in co-operation with the Oregon Agricultural College, the effect of a number of storage conditions was investigated. The concrete was of normal consistency; the aggregates were well graded sand and gravel. The mixture used in the Illinois test was 1:2:4,

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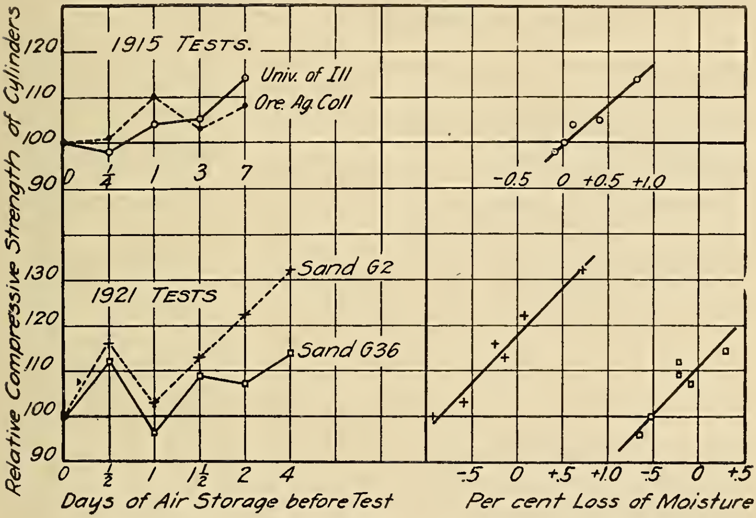
*Westergaard and Slater, "Moments and Stresses in Slabs," Proc. Am. Conc. Inst. v. 17, p. 476 (1921).

Mr. Richart.

in the Oregon tests, 1: 1.85: 4.15. In another series of tests made in 1921, particular attention was paid to the moisture content of the specimens throughout the tests. The concrete was approximately a 1: 2: 3½ mixture having 1.20 relative water content. Two artificially graded sands, 0-No. 50 and 0-No. 4 in size were used. The finer sand, though not of proper grading for a concrete aggregate, was used with the idea of producing a concrete especially affected by moisture changes. In both series, tests were made on five specimens of a kind and all were tested when 28 days old. The strength of the concrete cured in damp storage and tested damp was taken as the standard for comparison. The strengths under these standard conditions were: 1: 2: 4, 2900 lb. per sq. in., 1: 1.85: 4.14, 1760 lb. per sq. in., 1: 2: 3½, 0-No. 50 sand, 1870 lb. per sq. in., with 0-No. 4 sand, 2620 lb. per sq. in. Data of the tests are given in the accompanying table.



EFFECT OF MOISTURE CONTENT ON STRENGTH OF CONCRETE.

(Submitted by F. E. Richart.)

The diagram herewith shows the effect of moisture content in damp-cured specimens. In the specimens that were dried out immediately before testing, the length of the drying period does not appear so important as the loss of moisture, which was computed from the percent decrease in the weight of specimens from the time they were removed from the molds until tested. Some specimens apparently absorbed more water than other during the moist storage. The specimens dried in the oven at 120-140° F. showed the same relation between moisture content and strength as those dried in air, but the concrete exposed to the higher

Mr. Richart. temperatures appeared to be injured. The curves indicate that a moisture loss equal to 1% of the weight of the concrete produced an increase in strength of 17 to 20%, and that an increase of this magnitude is quite possible with favorable conditions for drying out the concrete.

Mr. Buente. W. S. BUENTE.—I notice that the maximum modulus of rupture is given in Prof. Abrams' test at about 600 lb. with some tests running about

EFFECT O STORAGE CONDITIONS AND MOISTURE CONTENT ON COMPRESSIVE STRENGTH OF CONCRETE.

(F. E. Richart.)

No.	Condition of Storage—Series of 1915.	Relative Strength.	
		Univ. of Ill.	Ore. Ag. Col.
1	*Standard.* Damp sand 28 days; tested damp	100	100
2	Damp sand 28 days; in air 6 hrs	98	102
3	Damp sand 27 days; in air 1 day	104	110
4	Damp sand 25 days; in air 3 days	105	103
5	Damp sand 21 days; in air 7 days	114	108
6	In form, top covered, 2 days; damp sand 14 days, air 12 days	109	111
7	In form, top covered, 28 days	102	100
8	In form, top exposed, 2 days; damp sand 26 days	98	99
9	In form, top exposed, 7 days; damp sand 21 days	98	95
10	In form, top exposed, 28 days	89	92
11	In air, under wet burlap, 28 days	87	95
12	In air of laboratory, 28 days	77	85
13	In air outdoors, 28 days	59*	87†

No.	Condition of Storage—Series of 1921.	Relative Strength.	
		Sand G 2 (O-50)	Sand G 36 (O-4)
14	*Standard.* Moist room, 28 days; tested damp	100	100
15	Moist room 27½ days; in air 12 hrs	116	112
16	Moist room 27 days; in air 1 day	103	96
17	Moist room 26½ days; in air 1½ days	113	109
18	Moist room 26 days; in air 2 days	122	107
19	Moist room 24 days; in air 4 days	132	114
20	Moist room 28 days; in oven at 140° F. 1 hr	118	106
21	Moist room 28 days, in oven at 120° F. 6 hrs	118	106
22	Moist room 28 days; in oven at 305° F. 3 hrs	94	83
23	Moist room 28 days; in oven at 270° F. 6 hrs	98	77
24	Air of laboratory, 28 days	86	67

* In November and December. † In July and August.

800. In the manufacture of plain concrete pipe to meet the test requirements of the A. S. T. M., the concrete must develop a modulus of rupture of about 1050 lb. The repeated tests made in plain concrete pipe, which show a modulus of rupture of the concrete as high as 1200, possibly 1500 lb., I would like to ask Prof. Abrams if he has made any tests on concrete rings that might explain this.

Mr. Abrams.

Prof. Abrams.—I will answer Mr. Buente's question first. The calculation of the modulus of rupture on curved sections as in concrete pipe is accompanied with many uncertainties. It must not be forgotten that the modulus of rupture is a good deal of a fiction after all. A great many assumptions are involved in calculating stresses in this way. If the concrete were entirely homogeneous and if there was a straight-line relation between stress and deformation up to the point of rupture, we would be on a good deal safer ground in calculating modulus of rupture and we could more properly compare the modulus of rupture of a straight beam with that of a curved beam; however, in the case of concrete which is a non-homogeneous material with a curvilinear relation between stress and deformation (of somewhat uncertain form) we have also all the complications that enter into the computation of stresses in a curved section.

I do not believe that the facts stated by Mr. Buente disprove any point brought out by our tests. I am aware that the stresses calculated from the loads required to produce failure in concrete pipes are higher than when the concrete is tested in a straight beam. In other words, the values do not represent actual differences in the quality of the concrete, but result in a measure from the inexact assumptions upon which such stresses are calculated. It should be noted also that the drier mixtures which the tamping or mechanical trowelling make it possible to use in concrete pipe will give concrete of much higher strength than the usual methods of placing concrete in road or building construction. These are conditions that cannot be reproduced readily in laboratory tests.

With reference to the question raised concerning Mr. Humphrey's tests. I think it will be clear that in a long beam the relation between stresses is much different from what it is in a very short beam. I had no intention of bringing up at this time a general discussion of *reinforced-concrete* members or structures. The tests on reinforced beams were thrown in incidentally. The study of the flexural strength of plain concrete and its relation to the compressive strength was the problems we were studying.

Mr. Richart's statement concerning the effect of drying out test specimens is an important item. That is why it is so important that in making test cylinders on the job, care should be taken to see that they are stored in a standard manner. If you do not do that, the results of the tests will vary considerably, due to differences in the curing condition, without any actual differences in the quality of the concrete.

Mr. Wight.

Frank C. Wight.—There is one point which might be brought up in reference to the curing condition of test pieces and its relation to the curing condition of the roads. I have found in traveling around the country that a good many road engineers are not disposed to consider the relation between a small prism or beam tested under certain curing conditions as exactly duplicated on a road. A road is a large slab with possibly moisture conditions under it, but not always, and under present

Mr. Wight. current methods, moisture conditions put on it afterwards during the curing period. In the northwest, particularly, I have found a number of road engineers who were much disposed to say that the small beams which are completely surrounded by moisture in the testing laboratory cannot be definitely related to a road slab. Many of them felt that there should be some method of test devised which would make more definite such relation. I think that is a point which the Research Committee might well consider.

Mr. Greulich. G. G. GREULICH.—I notice on some of the curves that the strength falls off rapidly yas the amount of water used in mixing is increased. Contractors like to use a lot of water for chuting, etc., if they can, and the argument is sometimes brought up that a test comes such a short time after a specimen is poured that excess-water specimens do show a weakness, but they claim that that weakness will wear off. Is the weakness caused by the separation of the aggregate when too much water is used, and is the decrease in the strength permanent, or does the excess of water injure the strength of the concrete only temporarily?

Mr. Abrams. PROF. ABRAMS.—The mixing water in concrete has two functions, (1) To hydrate the cement, and (2) to produce a plastic, mobile mass. The exact quantity of water necessary to hydrate the cement has not been accurately determined, but it seems probable that not more than 25 to 30% of the water usually placed in concrete is required for this purpose. When an excess of water is used, you are simply introducing a dispersing medium, in other words, the cement particles are separated just so much farther from each other than they were before. We do not know just how close together they were in the first place, but it is easy to calculate how much they are separated by adding more water. We know that if more and more water is added we soon come to a point where the cement never sets or hardens, although each particle goes through the process of hydration as it does in concrete. When water is added unnecessarily we are simply separating the cement particles and approaching a point where they will not set at all under ordinary conditions. This is shown by the fact that concrete strength drops off materially with small additions of water beyond that required to produce a plastic mass. In the tests reported, increasing the water-ratio (Fig. 9) from 0.80 to 1.20 (an increase of 50% in mixing water) we reduce the compressive strength from 2750 to about 950 lb. per sq. in.; and the modulus of rupture from 550 to 240 lb. per sq. in. In other words, we get a compressive strength of about 1/3 and a modulus of rupture of about 1/2 with the excess water, of that produced by a proper water content.

I should say, too, that the wettest consistency that we used in our tests does not represent the extremes to which concrete is frequently mixed

Mr. Abrams.

on building construction. No doubt all of you have seen concrete which would contain probably 50 percent more water than the maximum quantity we used; such concrete would give a strength probably 20 percent of the strength that would be produced by the same mixture under proper conditions.

With reference to the ultimate effect of this excess water, we have carried some compression tests to 1 and 2 years, and the indications are that this loss in strength is permanent. Very wet concrete will show a gain in strength which follows a curve exactly like the others when you plot age horizontally and strength vertically, but it will always be lower on the scale. There is no possibility that the wet concrete can ever attain the same strength as the drier, if both are cured and tested in the same way.

NOTE ON THE BALL TEST APPLIED TO CEMENT MORTAR.

By W. K. Hatt.*

It is well known that the ball test is a reliable index of the mechanical properties of material. This ball test has been applied by Brinell to metals and by Yanka and by the Forest Service to wood.

Recently in the Laboratory for Testing Materials of Purdue University the application of this test to mortar specimens has been investigated. This contribution describes the methods of making the test and preliminary results.

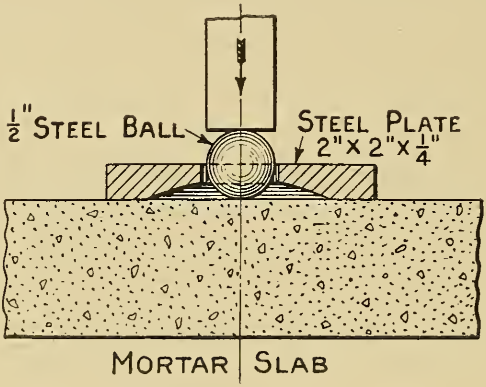


FIG. 1.—DIAGRAM SHOWING BALL TEST.

Methods of Test.

The ball is a standard steel ball ½ in. in diameter placed on the surface of a mortar slab which is 36x15x4 in. thick. A steel plate 2x2x¼ in. thick with a hole approximately ½ in. in diameter is placed over the ball which, therefore, projects half its diameter from the surface of the steel plate. The slab, ball, and the plate are placed on the platform of a testing machine and the load then applied on the projecting ball. This load is increased until the ball penetrates the surface of the mortar slab to one-half of its diameter, whereupon the testing head of the testing machine comes in contact with the surface of the steel plate. The resistance suddenly increases and throws up the scale beam which is a fairly good indication that the ball has penetrated the slab to one-half of its depth. For accurate results, however, deflectometers are placed on the slab to measure the penetration of the steel ball to the exact amount of

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*Professor of Civil Engineering, Purdue University, Lafayette, Ind.

½ in. The index of the test, therefore, is the load required to press the steel ball to one-half its diameter into the surface of the mortar.

The underside of this steel plate is slightly cut out to allow the material displaced by the ball to rise from the surface of the slab without moving the steel plate.

Of course this same test could be applied to the mortar in the surface of a concrete road or the floor of a building by using a lodometer or a hydraulic jack.

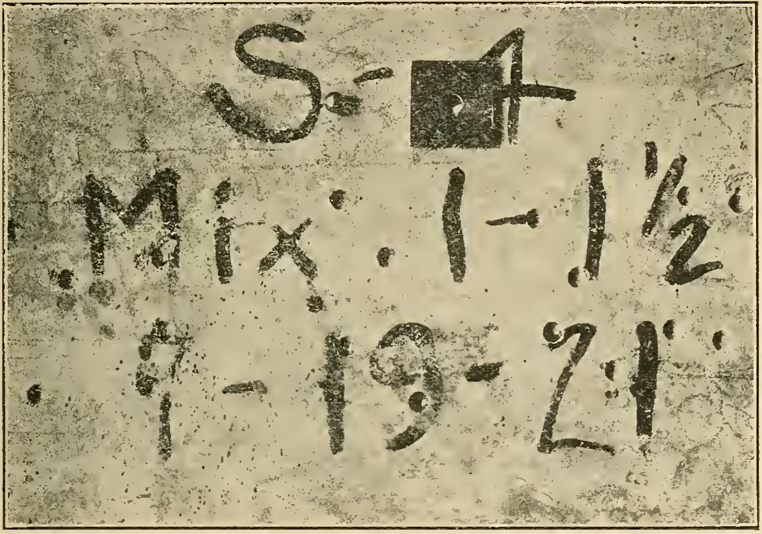

FIG. 2.—SOME RESULTS OF BALL TEST ON CONCRETE SURFACE.

Results of Tests.

Up to the present time the resistance of mortar slabs ranging from 1 : 1½ to 1 : 2½ mix has been determined at ages from 7 to about 40 days. Additional investigations have been made to determine the effect of a previously applied wheel load to the surface of the mortar. In these wheel tests a solid rubber tired truck wheel 34 in. in diameter with 4 in. width of tire has been applied to different portions of the slab with increasing loads up to 14,000 lb. The portions of the spots on the slab previously loaded with this wheel are then subjected to the ball test, which indicates the effect of these wheel loads upon the resistance of the concrete surfaces as determined by the subsequently applied ball test. These investigations of wheel loads have not progressed far enough at the present time to justify the publication of the data.

The results of the tests are shown in Tables I to V.

TABLE I.—BALL TEST OF CONCRETE.

Mark.	Mix of Mortar.	Age, days.	Load in lb. necessary to cause ½-in. ball to penetrate half its diameter.	
			Original Surface.	¼ in. below Original Surface.
S-7	1 : 1½	7	9,150	8,500
S-19		7	6,666	....
S-101		7	8,166	....
S-105		7	7,750	....
S-9		9	9,625	8,375
S-21		14	8,166	....
S-102		14	10,666	....
S-106		14	10,375	....
S-22		21	9,000	....
S-8		28	11,562	10,750
S-20		28	10,833	....
S-103		28	11,500	....
S-107		28	11,250	....
S-10		30	11,250	11,437
S-11	1 : 2	7	6,687	6,437
S-117		7	6,666	....
S-12		10	8,000	7,062
S-13		28	10,312	9,062
S-1		28	10,050	....
S-14		35	10,625	8,437
S-15	1 : 2½	7	5,062	5,187
S-16		10	6,000	5,166
S-17		28	7,062	6,875
S-18		42	9,250	8,437

NOTE.—Each value represents two or three ball impressions on one slab.

TABLE II.—COMPRESSIVE STRENGTH OF 6 x 12-IN. MORTAR CYLINDERS PARALLEL TO SLABS.

Mix.	Compressive strength in lb. per sq. in. for various ages, 7 to 42 days.								
	7	9	10	14	21	28	30	35	42
1 : 1½	2,739	4,110	....	3,755	4,199	4,773	4,687	....	....
	3,587	....	....	4,418	....	4,570	....	....	....
	3,681	....	....	3,595	....	3,993	....	....	....
	2,842	....	....	....	....	4,588	....	....	....
1 : 2	3,160	....	....	....	....	....	....	....	....
	2,255	....	....	....	....	4,011	....	4,276	....
1 : 2½	2,557	....	1,993	....	....	2,751	....	....	3,287

TABLE III.—BALL TEST OF CONCRETE.

Increased Resistance of Mortar Slabs at Different Ages.
(Average of four series including all mixes.)

Age in Days.	Relative Resistance, per cent.
7	70
14	85
21	95
28	100

See Table I for supporting data.

TABLE IV.—BALL TEST OF CONCRETE.

Relative Slab Resistance to Penetration of ½-in. Ball and Relative Compressive Strength of 6 x 12-in Cylinders for Different Mixes of Mortar. Based on 1 : 1½ Mortar. Age 28 days.

Mix.	Relative Compressive Strength of Concrete Cylinders.*				Relative Slab Resistance, Ball Test.†			
	Age, days.			Grand Average.	Age, days.			Grand Average.
	7	14	28		7	14	28	
1 : 1½	100	100	100	100	100	100	100	100
1 : 2	81	...	93	87	85	...	89	87
1 : 2½	76	57	66	66	65	...	69	67

* See Table II for supporting data.
† See Table I for supporting data.

TABLE V.—BALL TEST OF CONCRETE SUMMARY SHEET.

Mark.	Mix of Mortar.	Age, days.	Load necessary to cause ½-in. ball to penetrate one-half its diameter on original surface.									Number of Applications of Initial Load.
			Initial Load on Wheel (lb.).									
			000	400	800	1,600	2,400	3,200	4,000	4,800	5,600	
S-101	1 : 1½	7	8,166	7,375	7,250	6,875	6,250	6,000	5,500	5,500	5,250	1
S-102	1 : 1½	14	10,666	8,750	8,250	8,375	7,625	7,750	7,000	6,750	6,750	1
S-103	1 : 1½	28	11,500	8,250	8,500	8,000	7,125	6,875	6,625	6,125	6,675	1
S-105	1 : 1½	7	7,750	5,750	5,750	5,750	5,000	5,125	4,875	5,000	4,750	25
S-106	1 : 1½	14	10,375	7,625	7,000	6,500	6,250	6,250	5,750	5,500	4,750	25
S-107	1 : 1½	28	11,250	9,250	8,125	7,875	7,500	7,250	7,000	6,500	6,125	25
S-117	1 : 2	7	6,666	5,250	5,000	4,875	4,750	4,500	4,500	4,250	4,000	1
S-118	1 : 2	14	7,750	6,750	6,375	6,000	5,500	5,375	5,250	5,125	5,000	1
S-119	1 : 2	28	9,250	7,750	7,250	7,000	6,750	6,500	6,375	6,250	5,875	1
S-121	1 : 2	7	7,000	5,875	5,500	5,125	4,500	4,500	4,250	4,000	4,000	25

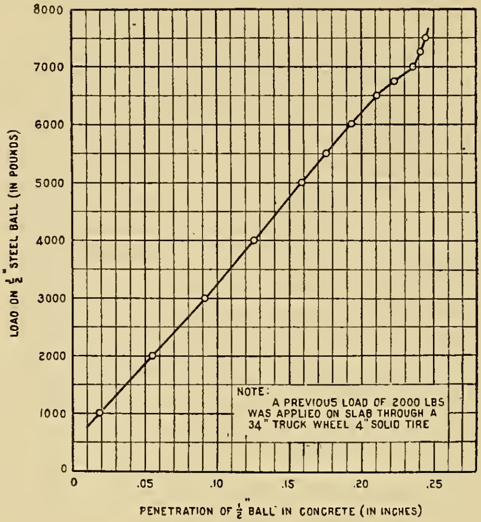

FIG. 3.—PENETRATION LOAD CURVE.

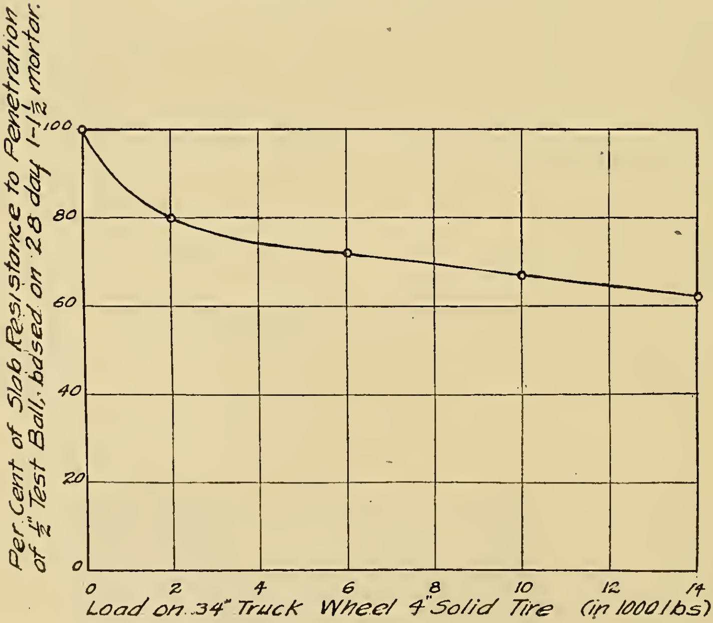

FIG. 4.—RESISTANCE OF MORTAR TO PENETRATION OF ½-IN. TEST BALL.

For Various Wheel Loads Applied on a 34-in. 4 in. Solid Tire Truck Wheel.

CONCLUSIONS.

This test is simple in its application and should have a wide range of usefulness. Results show that this test indicates some mechanical property of the material and exhibits the same general relations as are determined by the more complicated expensive compression and tension tests of mortar.

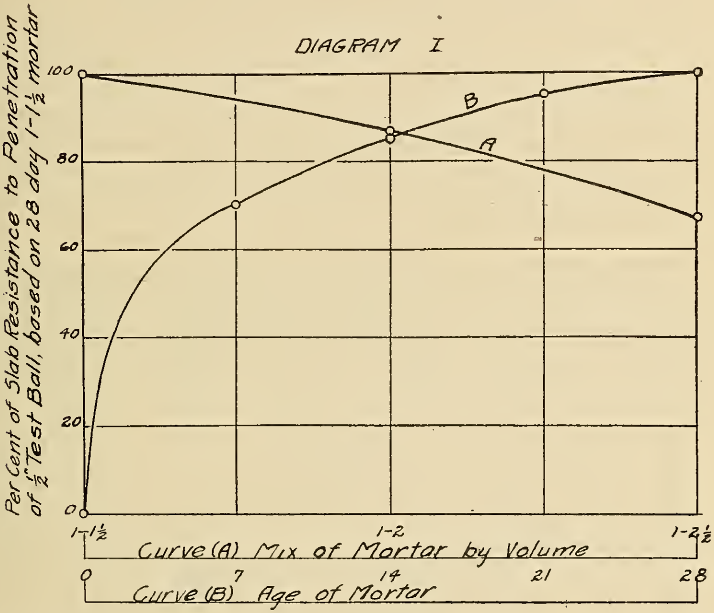

FIG. 5.—RESISTANCE OF MORTARS FOR ½-IN. TEST BALL UNDER VARIOUS CONDITIONS.

A. Mixtures of 1 : 1½, 1 : 2, 1 : 2½ Mortars, Age 28 Days.
B. Various Ages up to 1 Mo. for Averages of Mixes 1 : 1½, 1 : 2, 1 : 2½.

Of course if the ball test is applied to concrete the ball should be pushed into the mortar and should not rest upon a piece of large aggregate.

AN ACCURATE MECHANICAL SOLUTION OF STATICALLY INDETERMINATE STRUCTURES BY USE OF PAPER MODELS AND SPECIAL GAGES.

By George Erle Beggs*

During the summer of 1916 my attention was given to the problem of the deflections and stresses of the triple span continuous trusses of the new B. & L. E. R. R. bridge over the Allegheny River. One of these trusses is shown in Fig. 1. On account of the various conditions of loading for which it was necessary to calculate the four-truss reactions, it was considered advantageous to construct their influence lines, after calculating their ordinates by the theories of deflection and least work. These influence lines, plotted in Fig. 1, resembled the elastic curves of continuous beams fastened at three supports and deflected a unit distance at the remaining support

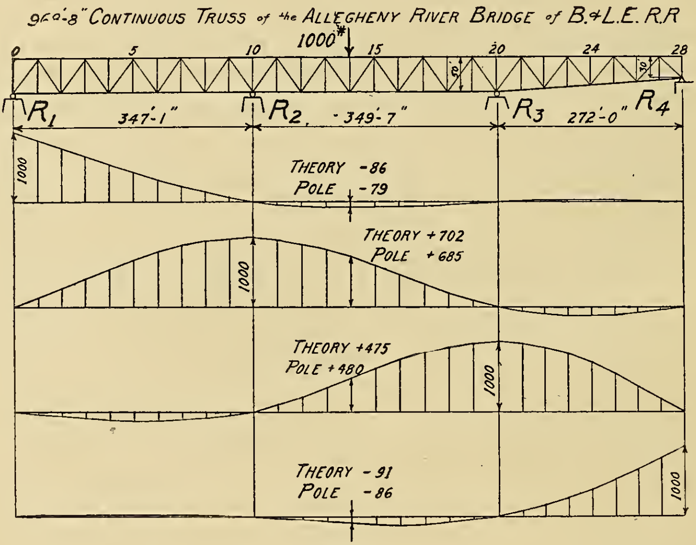

FIG. 1.

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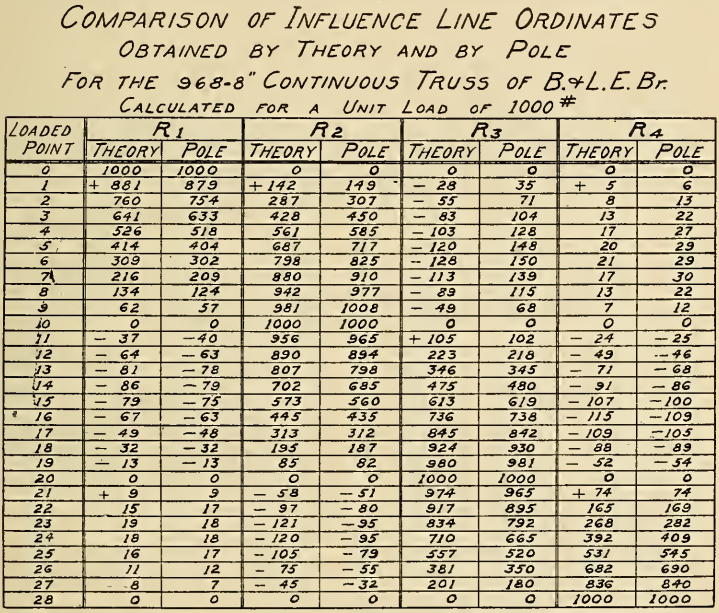
*Associate Professor of Civil Engineering, Princeton University, Princeton, N. J.

for whose reaction the influence line was drawn. To determine the accuracy of this observation, a 10-ft. pole of uniform section and three nails were used. To draw the influence line for R_1 the pole was restrained by single nails at R_2, R_3, R_4, and deflected at R_1, a distance to scale 1000 lb. The ordinates to this elastic curve were measured.

The comparison of values obtained by this crude mechanical apparatus and by exact theory are given in Fig. 2. The agreement of values is seen to be remarkably good.

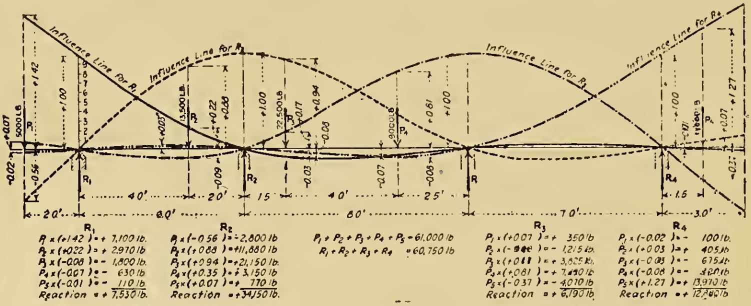
Three years later, in the summer of 1919, the attention of R. Fleming of the American Bridge Co. was called to this method of solution, with the result that he carefully computed the reactions of the three-span continuous beam of uniform section shown in Fig. 3, and published the solution in the *Engineering News-Record,* August 28, 1919. For the mechanical method he used a 3-ft. spline with ordinary pins for supports. (Fig. 3.) The influence lines were assumed to correspond to the elastic curves produced by bending the spline a distance unity from each of the four supports in succession as shown. The reactions for the actual loadings were then found by summing algebraically the loads times their corresponding ordinates. The closeness with which the reactions sum to the total load is remarkable and the comparison between reactions obtained by theory and by spline is practically satisfactory.


COMPARISON OF INFLUENCE LINE ORDINATES
OBTAINED BY THEORY AND BY POLE
FOR THE 968-8" CONTINUOUS TRUSS OF B.&L.E. BR.
CALCULATED FOR A UNIT LOAD OF 1000#

LOADED POINT	R1		R2		R3		R4	
	THEORY	POLE	THEORY	POLE	THEORY	POLE	THEORY	POLE
0	1000	1000	0	0	0	0	0	0
1	+ 881	879	+ 142	149	− 28	35	+ 5	6
2	760	754	287	307	− 55	71	8	13
3	641	633	428	450	− 83	104	13	22
4	526	518	561	585	− 103	128	17	27
5	414	404	687	717	− 120	148	20	29
6	309	302	798	825	− 128	150	21	29
7	216	209	880	910	− 113	139	17	30
8	134	124	942	977	− 89	115	13	22
9	62	57	981	1008	− 49	68	7	12
10	0	0	1000	1000	0	0	0	0
11	− 37	− 40	956	965	+ 105	102	− 24	− 25
12	− 64	− 63	890	894	223	218	− 49	− 46
13	− 81	− 78	807	798	346	345	− 71	− 68
14	− 86	− 79	702	685	475	480	− 91	− 86
15	− 79	− 75	573	560	613	619	− 107	− 100
16	− 67	− 63	445	435	736	738	− 115	− 109
17	− 49	− 48	313	312	845	842	− 109	− 105
18	− 32	− 32	195	187	924	930	− 88	− 89
19	− 13	− 13	85	82	980	981	− 52	− 54
20	0	0	0	0	1000	1000	0	0
21	+ 9	9	− 58	− 51	974	965	+ 74	74
22	15	17	− 97	− 80	917	895	165	169
23	19	18	− 121	− 95	834	792	268	282
24	18	18	− 120	− 95	710	665	392	409
25	16	17	− 105	− 79	557	520	531	545
26	11	12	− 75	− 55	381	350	682	690
27	8	7	− 45	− 32	201	180	836	840
28	0	0	0	0	0	0	1000	1000

FIG. 2.

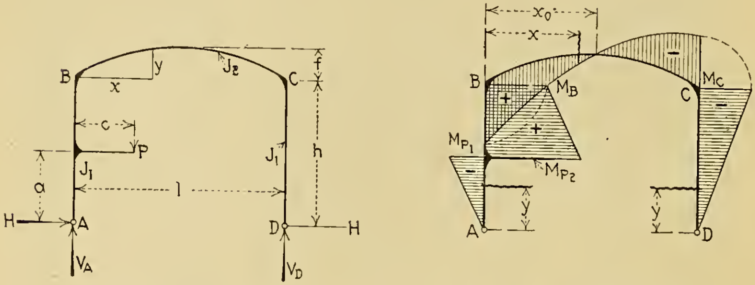
To Mr. Fleming, at that time, I stated the possibility of extending this mechanical method to the solution of indeterminate frames. Does not the layman estimate the severity of stress in an elastic structure by observation of its distortion under load,—the "give and swell" of the members as one man expressed it? Why then should not the engineer closely observe the deflections of model elastic structures, and so estimate their reactions, moments, shears, and thrusts, and thus avoid the derivation or the application of such laborious and questionable formulas as quoted in Figs. 4 and 5 from German and American publications?



Theory 7,392 lb. *Theory 34,304 lb.* *Theory 6,782 lb.* *Theory 12,524 lb.*

REACTION FOR CONTINUOUS BEAM BY MECHANICALLY-DRAWN INFLUENCE LINES

FIG. 3.



$$V_A = P \cdot \frac{l-c}{l}; \quad V_D = P \cdot \frac{c}{l}; \quad y = \frac{4f}{l^2} \cdot x\,(l-x); \quad k = \frac{J_2}{J_1} \cdot \frac{h}{l}$$

$$H = \frac{5\,Pc}{2\,h} \cdot \frac{3\,k(h^2-a^2)+h(3\,h+2f)}{5\,h^2\,(2\,k+3)+4f(5\,h+2f)}.$$

Momente

$$M_{P_1} = -\,H\,a$$

$$M_{P_2} = +\,P\,c - H\,a$$

$$M_B = +\,P\,c - H\,h.$$

im Querriegel (Abszisse x und Ordinate y auf die Achse B—C bezogen);

$$M_x = +P \cdot \frac{c}{l}\,(l-x) - H\,(h+y)$$

$$M_C = -H\,h$$

FIG. 4.

To determine the value of V_b in Fig. 5, in the case where the loading on the three spans is not symmetrical, requires by the well known method of slope deflections the accurate solution of nine equations with nine unknowns, and a very considerable amount of work in addition thereto. Such tedious calculations are most disheartening, and lead the so-called "practical man" to make his best guess. Nature solves such problems in actual structures instantly and accurately. Could not the engineer enlist nature

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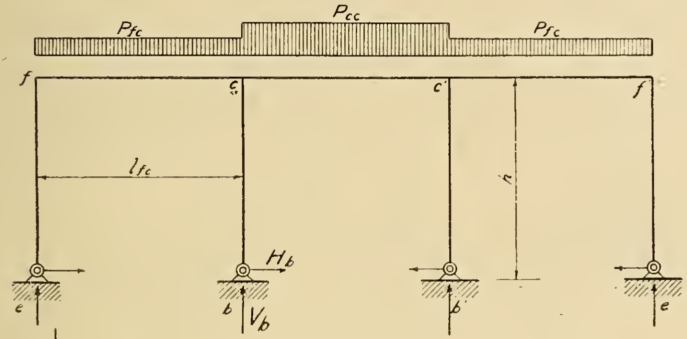


$$V_b = \frac{P_{fc}\, l_{fc}\left[9+15\frac{hI_{fc}}{l_{fc}I_{ef}}+9\frac{hI_{fc}}{l_{fc}I_{bc}}+6\frac{hI_{cc}}{l_{cc}I_{bc}}+2\frac{hI_{fc}}{l_{fc}I_{ef}}\left(6\frac{hI_{fc}}{l_{fc}I_{bc}}+12\frac{hI_{cc}}{l_{cc}I_{bc}}\right)\right]+P_{cc}\, l_{cc}\left[9+12\frac{hI_{fc}}{l_{fc}I_{ef}}+12\frac{hI_{fc}}{l_{fc}I_{bc}}+6\frac{hI_{cc}}{l_{cc}I_{bc}}+2\frac{hI_{fc}}{l_{fc}I_{ef}}\left(6\frac{hI_{fc}}{l_{fc}I_{bc}}+4\frac{hI_{cc}}{l_{cc}I_{bc}}\right)+\frac{hI_{cc}I_{fc}}{l_{fc}I_{bc}}\left(3+2\frac{hI_{fc}}{l_{fc}I_{ef}}\right)\right]}{18+24\frac{hI_{fc}}{l_{fc}I_{ef}}+24\frac{hI_{fc}}{l_{fc}I_{bc}}+12\frac{hI_{cc}}{l_{cc}I_{bc}}+4\frac{hI_{fc}}{l_{fc}I_{ef}}\left(6\frac{hI_{fc}}{l_{fc}I_{bc}}+4\frac{hI_{cc}}{l_{cc}I_{bc}}\right)}$$

FIG. 5.

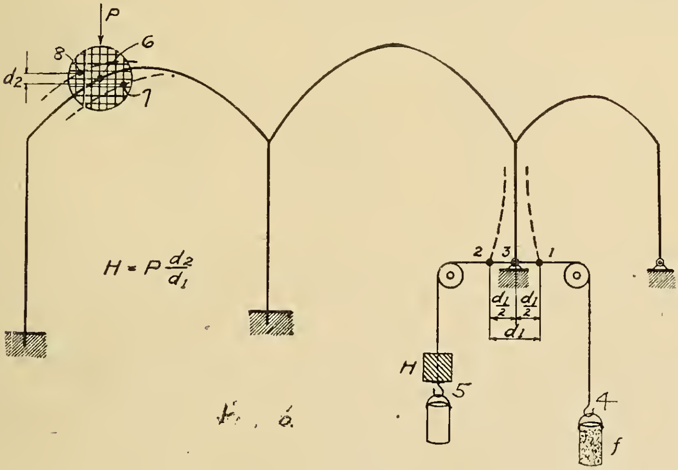


FIG. 6.

as his ally, and so eliminate much of the mathematical work incident to the solution of problems in indeterminate structures?

The fundamental principle of any mechanical solution is Maxwell's Theorem of Reciprocal Deflections, as proved and applied in all texts on higher structures. Consider the very complicated structure shown in Fig. 6. Suppose in the first instance that this figure represents the actual full-sized structure. Let it be required to find the horizontal thrust at the hinge at the foot of the third column caused by a load *P* on the first arch. Assume for the sake of argument that the third support is changed from hinged to roller bearing, so that horizontal motion may occur freely. Attach to the lower end of the column a cable which carries a weight *H*. This weight is such that it holds the column from deflecting horizontally when the load *P* rests on the arch. Now attach to the right-hand cable shown a pail carrying a very small load *f* pounds of sand. From the weight *H* hang an empty pail. The foot of column deflects a small distance $0.5\ d_1$ from point *3* to point *1*, and at the same time the load *P* is moved from point *6* to some point 7. Now transfer the sand *f* to the pail at left of *3*. The column deflects a horizontal distance d_1 from *1* to *2*, while the load *P* moves from point 7 to 8. From Maxwell's Theorem, it follows that

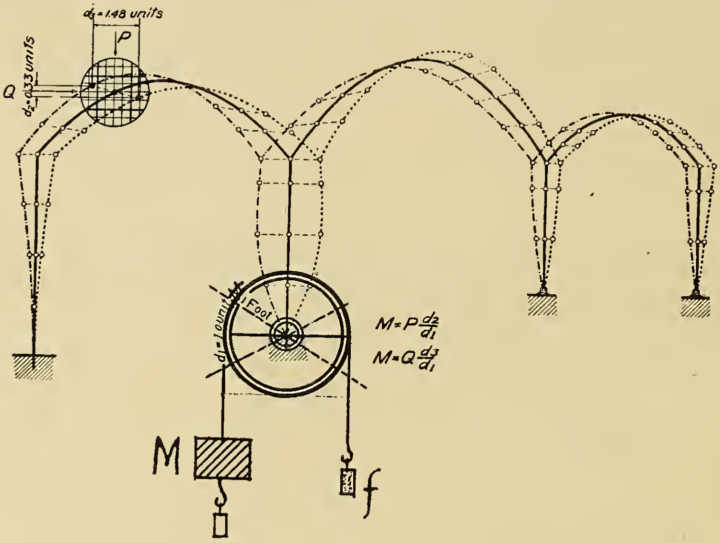


FIG. 7.

$$H \text{ equals } P\left(\frac{d_2}{d_1}\right)$$

It is to be noted that in general the value d_2 is the component of the deflection of *P* in the direction of *P*. It is evident from this equation that *H* becomes known as soon as the ratio d_2/d_1 is evaluated, and the load P specified. It is further apparent that this ratio of deflections may be obtained by applying to an elastic unloaded model of the actual structure a small deflection d_1 and measuring by accurate means the corresponding deflection d_2. There is an advantage in introducing the deflection d_1, half to the right and half to the left of the mean position, for the frame is distorted from its geometric position less than if the deflection were all on the same side of the mean geometric position.

Assume in the second instance that it is desired to know the bending moment at the foot of the second column due to a load P on the first arch.

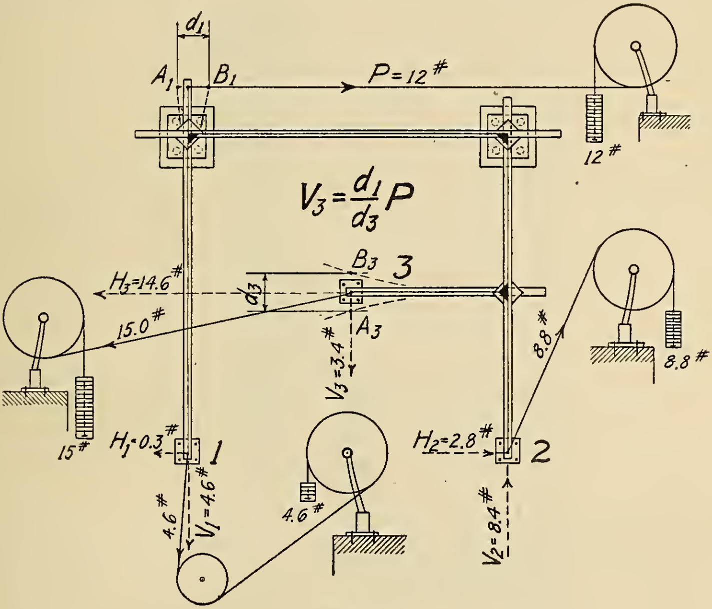


FIG. 8.

(Fig. 7.) Assume that the connection of this second column is changed from fixed to hinged. Let the hinge pin be a shaft keyed to the foot of this column and turning freely in a bearing that is fixed to the foundation. On this same shaft mount a wheel of 1 ft. radius as shown. Support by a cable wound on the circumference of this wheel a weight M of such value that the foot of this column is not rotated from its normal position when the load P rests on the first arch. Attach to the right-hand cable a pail carrying a very small load f pounds of sand, and from the weight M, hang an empty pail as shown. The circumference of the wheel deflects a small distance $0.5\, d_1$ clockwise, and at the same time the load P is moved downward and toward the right. The motion shown by the drawing is greatly magnified. Now transfer the sand f to the empty pail. The circumference of the wheel will deflect a distance d_1 counter clockwise, and at the same time the load P will move upward and toward the left, the component of its motion in the direction of P being d_2. From Maxwell's Theorem it follows that

$$M = P\,d_2/d_1.$$

It is evident that the value of the weight M that will produce elastic equilibrium becomes known as soon as the ratio of d_2 to d_1 is evaluated and the load P specified. It is also apparent that this ratio may be obtained by applying to an elastic unloaded model of this structure a corresponding small rotative deflection d_1 and measuring by accurate means the corresponding deflection d_2.

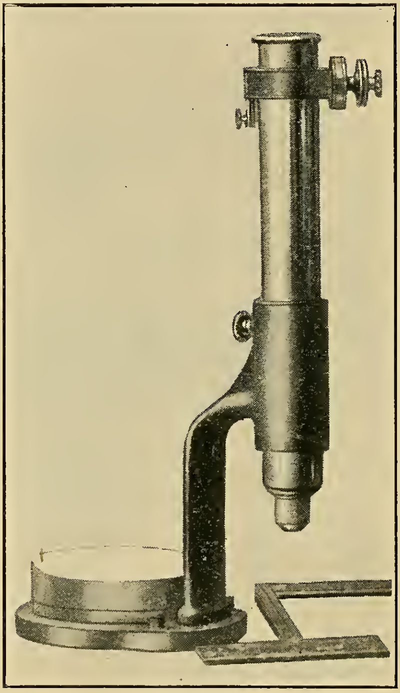
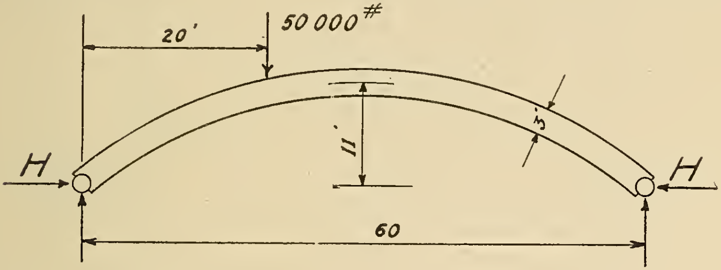


FIG. 9.

It is further clear that the value of the weight M is numerically equal to the bending moment at the foot of the second column, since the radius of the wheel is unity. As before and for the same reason, the deflection d_1 is applied half on either side from the mean geometric position.

Not wishing to rely entirely on such philosophical considerations, I decided in January, 1920, to check the above principle by a practical test.

A model of an indeterminate structure of the form shown in Fig. 8 was constructed. The frame was made of wood strips about ⅝ by ⅞ in., the columns and main horizontal member being 50 in. in length center to center of connections. Metal gussets and screws fastened the bars together. The three-hinged frame was supported in a horizontal position on the floor and floated on ball bearings that moved freely between glass plates. While measuring deflections no weights were applied to the structure. In order to determine by deflections the value of V_3 at support *3* that would be caused by a load P equal to 12 lb., the ratio d_1/d_3 was first determined as follows: the pin at *3* was removed and the point *3* of frame forced to deflect vertically upward by an amount $0.5\,d_3$ and then vertically downward by the same amount. The corresponding motion of point B_1 was to A_1, and its deflection d_1 was measured. According to the principle previously explained, V_3 equals d_1/d_3 12 lb., in which equation the ratio of the known values d_1 and d_3 was such that V_3 was calculated to be 3.4 lb. By similar method H_3 was calculated to be 14.6 lb. The hinge reaction at 3 for P equal to 12 lb. was thus completely determined, its value 15 lb. being equal to



H	6" Celluloid Model	Theory
	44600	44800

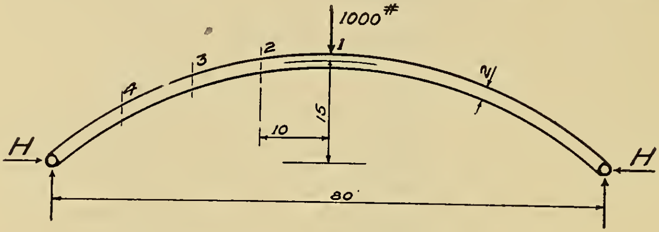
FIG. 10.

$$\sqrt{14.6^2 + 3.4^2}.$$

The pin was replaced in hinge *3*. The load P equal to 12 lb. was applied by means of a counterweight acting through a cord wound over a bicycle wheel as shown in Fig. 8. A force of 15 lb. was similarly applied in the line of the resultant of H_3 and V_3 by a cord attached to a screw in the bar close to point *3*. The pin at hinge *3* was then removed. To my great satisfaction no deflection occurred upon removal of this pin. The reaction of 15 lb. at hinge *3* as determined mechanically, was therefore correct, for it produced both static and elastic equilibrium. Similarly the reactions at hinges *1* and *2* were determined, and their correctness tested. The three reactions as determined entirely by deflection measurements are seen to satisfy very well the three laws of statical equilibrium. The small errors may be chargeable to the crudeness of the apparatus.

In order to reduce the scale of the models and at the same time to improve the accuracy of the observations, it was soon decided to make the models of celluloid, and to read the deflections by use of micrometer microscopes of the type shown in Fig. 9. This microscope will read directly to about 1/16000 of an inch. It is a stock pattern except for the cross section lining in the field of view.

The first structure solved by aid of the microscope was the arch shown in Fig. 10. A celluloid model of this arch to scale span equals 6 in. was placed on the drawing board. The right hinge pin was made immovable,

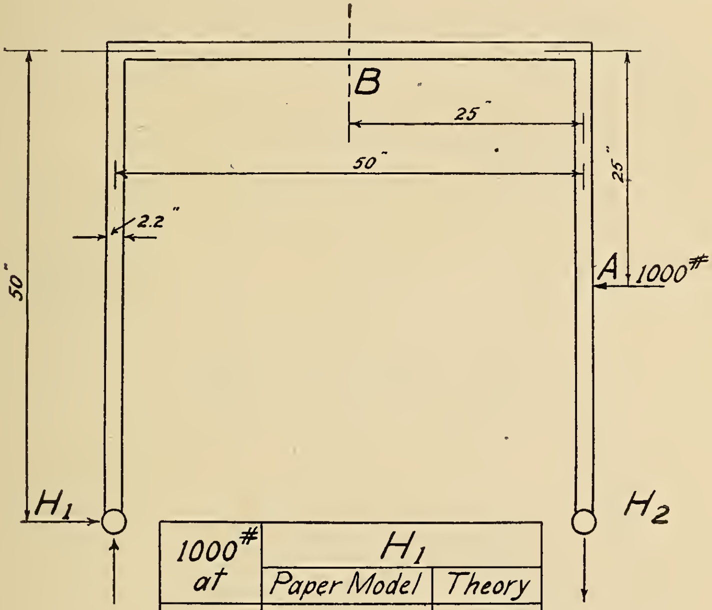


1000# at	H	
	Model	Theory
1	1000	1000
2	920	930
3	720	720
4	420	400

FIG. 11.

and the left hinge pin was allowed horizontal motion. The arch floated on small ball bearings placed between it and the drawing board. One microscope was placed over the left hinge and oriented to read its horizontal motion. The other microscope was placed over the point of application of the vertical load and oriented to read the *vertical component* of the deflection of the load point. The experiment was most encouraging. The ratio of the measured vertical deflection of the load point to the horizontal motion of hinge multiplied by 50,000 gave *H* equal to 44,600 lb. as compared to 44,800 given by the common theory.

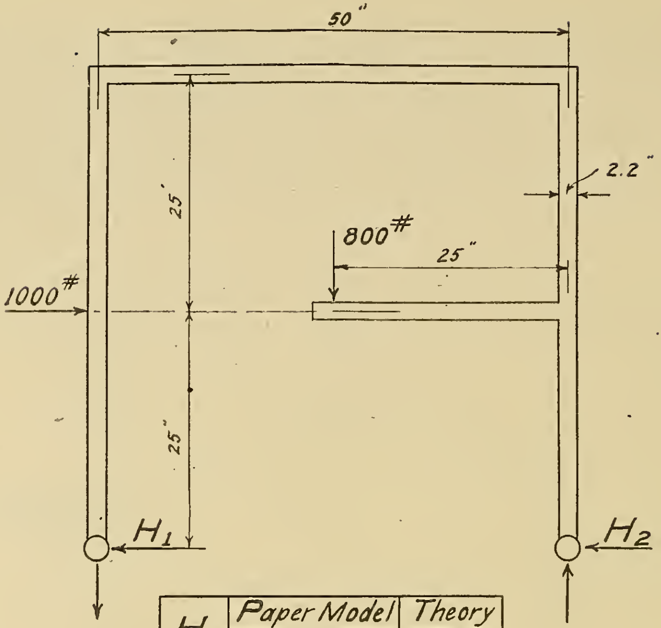
The practical difficulty of the method was the making of the celluloid models. At the suggestion of one of my students, I decided to use heavy paper or cardboard for model making. The thickness chosen can be cut with a few heavy strokes of the penknife. Needles pressed through the paper make very satisfactory hinges. A model of the two-hinged arch shown in Fig. 11 was made of paper to scale span equal to 20 in. The values of H obtained from the deflections of this paper model read through the microscope compare very closely with those given by the common theory. Models of the structures shown in the following six illustrations were also made of thick paper, and the reactions found by deflection readings are seen to agree with theory as closely as might be expected. In Figs. 12 and 13 the agreement is very close. In Fig. 14 the value of H obtained from the model is smaller than H by theory. It is believed that the model value is nearer the truth. A little consideration will show that as the depth of the rib increases, the value of H should decrease, until the reactions become almost entirely vertical when the rib is several feet deep. The effect of the increase of the depth of rib, on the reduction of H is shown by the results in Fig. 15.



1000# at	H1 Paper Model	H1 Theory
A	283	287
B	81	75
A and B	364	362

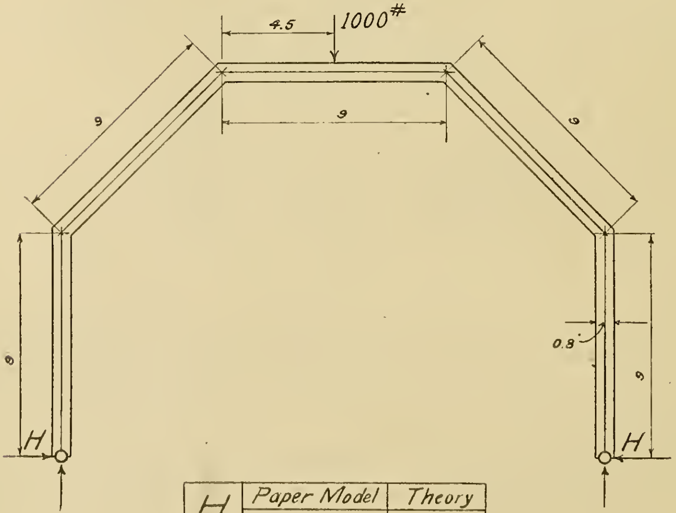
FIG. 12.

The difference between results by model and theory for the ring in Fig. 16 may be partly chargeable to the approximations in the mathematical analysis. In any case the error is negligible for practical purposes.



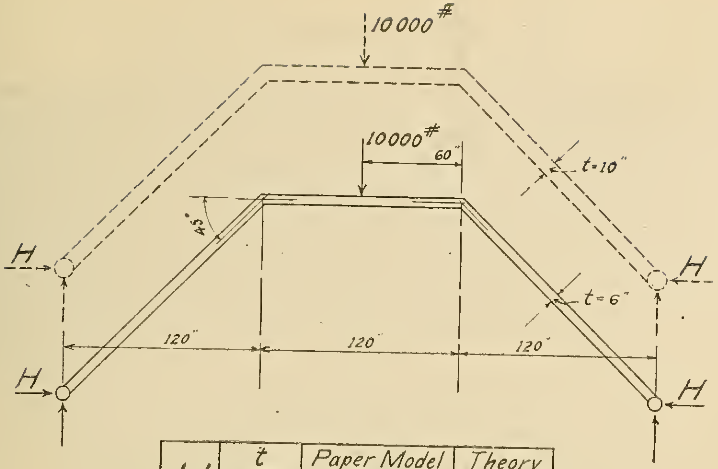
H_2	Paper Model	Theory
	497.4	497

FIG. 13.



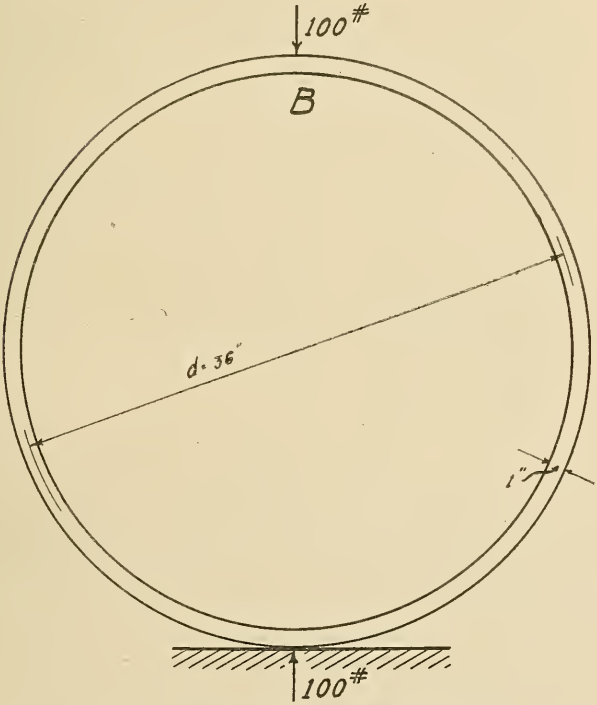
H	Paper Model	Theory
	1440	1530

FIG. 14.



	t	Paper Model	Theory
H	6"	5530	5637
	10"	5430	5637

FIG. 15.



M_B	Paper Model	Theory
	590	572.4

FIG. 16.

The paper model of the arch with fixed ends shown in Fig. 17 was made to scale span equal to 20 in. The right end was fastened to the drawing board and the left end glued to a movable block. The model was floated on small ball bearings. Unit, moment, thrust, and shear deformations were introduced in succession at the left support, where one micrometer microscope was used to measure the motion of the foundation. The second microscope was used to read the vertical components of the deflections of the rib. These latter deflections are directly proportional to the influence line ordinates for the moment, shear, and thrust at the left support of the arch. The small variation between the dotted and full influence lines in Fig. 17 indicates the close agreement of the values read from the paper model with those obtained by theory. A record of the numerical values by the two methods is given in Fig. 18.

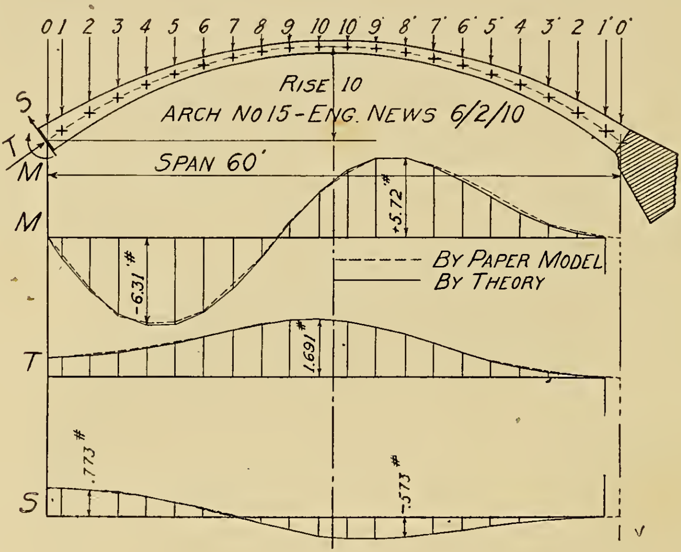


FIG. 17.

In order to bring the theory into agreement with the results read from the model, it was necessary to apply to the values obtained by theory the correction for thrust shortening. The two solutions agree quite closely enough for practical design. There are strong reasons for believing that the mechanical solution is fully as near the truth as is the theoretical solution.

The final development in the mechanical method was the design and application of the deformeter gage shown in Fig. 19. The function of this gage is to introduce at any section of the model, either at the support or at any other section of the structure, a unit deformation. This may be a pure thrust, shear, or moment deformation. Suppose, for example, that it is desired to obtain the thrust at the fixed end of some member M of any indeterminate structure due to a load P applied at some point A of the same structure and acting in the direction CA. These latter points appear in the field of the micrometer microscope set up over some load point A of the model structure. The cross lines of the microscope field are oriented in the direction CA of the applied load P which direction is pencilled on the model for convenience in orienting the microscope. At the fixed support the paper model M is clamped to the gage by means of the plate d. The model floats freely on small ball bearings. The bar a of the gage is fastened to the drawing board by means of the phonograph needles n, a thin piece of paper f being used between a and the drawing board so that the bar b may have clearance and free motion. For hinged supports the plate d is omitted, and the pin h in bar b acts as the hinge pin. If the thrust at the fixed support due to a load P acting in direction CA at A is desired, remove the two round normal plugs in the deformeter gage with two small wooden wedges w. Place in the gage the larger pair of thrust plugs shown. Read in the microscope the position of the load point A of the model. Next place the pair of smaller thrust plugs in the gage. The load point A is seen in the microscope to move along a straight line to a point B. Take reading of the position B, and by difference obtain the deflection $AC = d_2$. If the

COMPARISON OF VALUES OBTAINED BY THEORY AND BY PAPER MODEL
FOR ARCH NO. 15 ENG. NEWS 6-2-10
EFFECT OF THRUST SHORTENING INCLUDED IN BOTH CASES

LOADED POINT	MOMENT		THRUST		SHEAR	
	THEORY	MODEL	THEORY	MODEL	THEORY	MODEL
0	−0.00	−0.00	+.584	+.584	+.807	+.807
1	−1.50	−1.38	.585	.590	.810	.780
2	−3.86	−3.70	.626	.660	.773	.785
3	−5.49	−5.60	.706	.737	.700	.680
4	−6.31	−6.18	.828	.848	.586	.588
5	−6.24	−6.02	.982	1.015	.430	.420
6	−5.30	−5.28	1.168	1.175	.240	.219
7	−3.63	−3.33	1.353	1.365	.300	.030
8	−1.40	−1.29	1.515	1.540	−.182	−.191
9	+1.04	+1.17	1.645	1.635	−.388	−.364
10	3.25	3.13	1.691	1.675	−.539	−.522
10'	4.87	4.64	1.628	1.625	−.627	−.610
9'	5.67	5.70	1.466	1.470	−.632	−.612
8'	5.72	5.71	1.232	1.245	−.573	−.570
7'	5.06	5.14	.980	.990	−.486	−.494
6'	4.07	4.24	.726	.750	−.375	−.383
5'	2.89	3.19	.482	.503	−.260	−.272
4'	1.80	2.00	.285	.325	−.159	−.177
3'	0.93	1.00	.140	.175	−.080	−.089
2'	0.31	0.51	.043	.061	−.028	−.033
1'	0.00	0.03	.000	.004	−.000	−.007
0'	0.00	0.00	.000	.000	−.000	−.000

FIG. 18.

total thrust deformation introduced by the standard deformeter gage be d_1, the value of the thrust at the support in question equals $(d_2/d_1)\ P$, and is a compression, for the upward deflection at the support has caused the load point A to deflect in a direction opposite to the direction CA of the applied load. The mechanical method therefore gives both the sign and value of the desired component of the reaction. To obtain the moment and shear at the same section, the moment and shear plugs are used in the gage in similar manner and values of moment and shear are calculated from the two corresponding deflection measurements made with the microscope.

If the moment, shear, and thrust at some interior section of an internally indeterminate structure must be found, the gage is clamped at such section by means of two bars d, one as shown in Fig. 19, and another bar d by screws in holes g. The gage in this case floats on three ball bearings which move freely between two thin glass plates. The model is cut at this section. The three sets of plugs for moment, shear, and thrust are then introduced in succession, the corresponding deflections of the load points of the model are read through the microscope, and the corresponding moment, thrust, and shear at the section evaluated as before. The values shown in the following six illustrations were obtained by use of small scale paper models. (Scale 1 in. = 1 ft.) The deflections were introduced by the standard deformeter gages and the deflection of the load points read through the microscope.

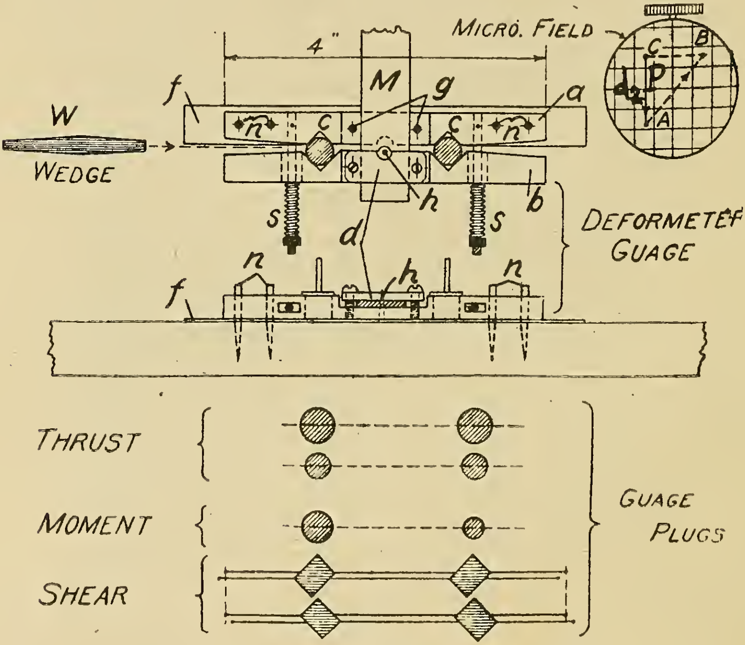
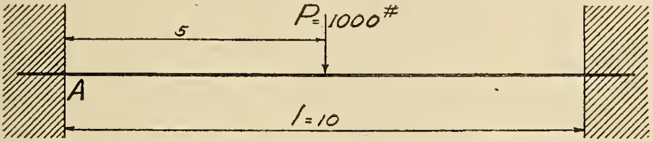


FIG. 19.

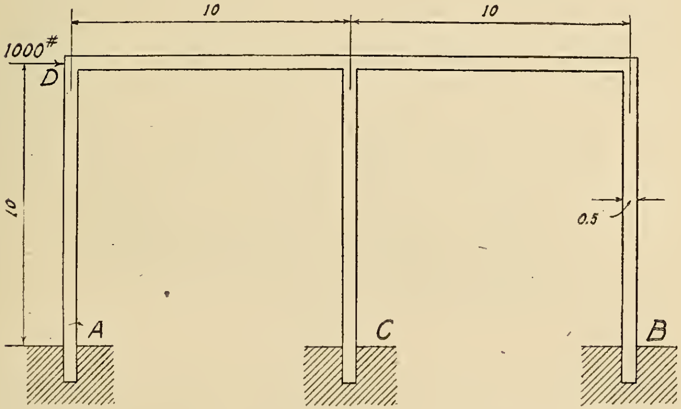
In Fig. 20 the deformeter gage was attached at A and the microscope oriented to read vertical deflection at the center of span caused by the small rotation at A produced by reversal of the moment plugs in the gage. The agreement of the two results for moment at the support is satisfactory.



M_A

Paper Model	1230'#
Theory Pl/8	1250'#

FIG. 20.



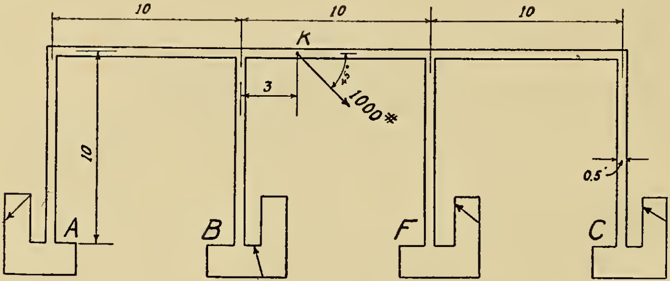
	Paper Model				Theory		
	A	C	B	Ck.	A	C	B
H	301 ←	380 ←	309 ←	$\Sigma H = 10$	302	396	302
V	238 ↓	10 ↑	225 ↑	$\Sigma V = 3$	219	0	219
M	1715 ↻	1935 ↻	1720 ↻	$\Sigma M_D = 30$ ↺	1771	2084	1771

FIG. 21.

In Fig. 21, the deformeter gages were attached at the three supports A, C, B, and the three components of reactions determined by reading the horizontal deflections of model at the load point D. The forces and couples at A, C, B, so determined together with the load at D are found to satisfy the three equations of equilibrium with remarkable precision.

The components of the reactions of the four-post portal in Fig. 22 were similarly determined both by model and by slope deflections.

The reactions calculated by the model deflections again satisfy very well the three laws of equilibrium, and agree reasonably well with the values obtained by theory. There is no reason to believe that the theory values are more accurate than those obtained by the model. From the components of these four reactions, their magnitudes, directions, and positions were calculated. A celluloid model of the structure was then made to a scale 1 in. = 1 ft. The lines of the reactions as determined from the deflections of the paper model were scribed on the extended foundations of this celluloid model as shown in Fig. 22. Holes were drilled along these lines and a single hinge pin put in each foundation anywhere along the line of its reaction. A varying force was then applied at the point *K* in the direction shown, causing the structure to deflect throughout. None of the four foundations rotated appreciably on their single hinge pins. This is a practical proof of the correctness of position and direction of the reactions obtained by the deflections of the paper model.



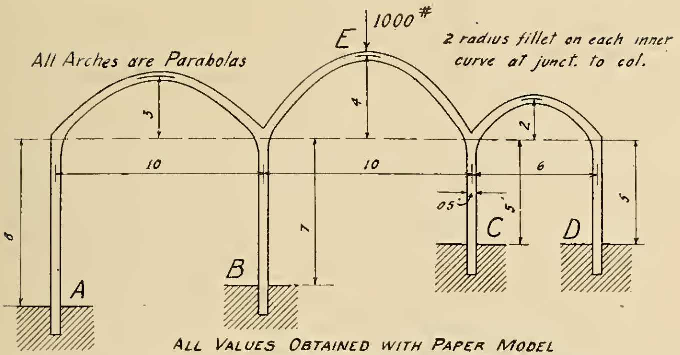
A . B . F . C FIXED

	Paper Model			Theory		
	H	V	M	H	V	M
A	168 ←	173 ↓	915)	176	165	992
B	150 ←	630 ↑	815)	142	611	877
F	206 ←	148 ↑	1000)	236	164	1189
C	165 ←	106 ↑	945)	153	96	914
Ck.	$\Sigma H = 18$	$\Sigma V = 4$	$M_K = 18$)			

FIG. 22.

The celluloid model of this particular structure is on exhibit. Also the microscopes, gages, and a few of the small celluloid and paper models are here for the inspection of those who may be interested.

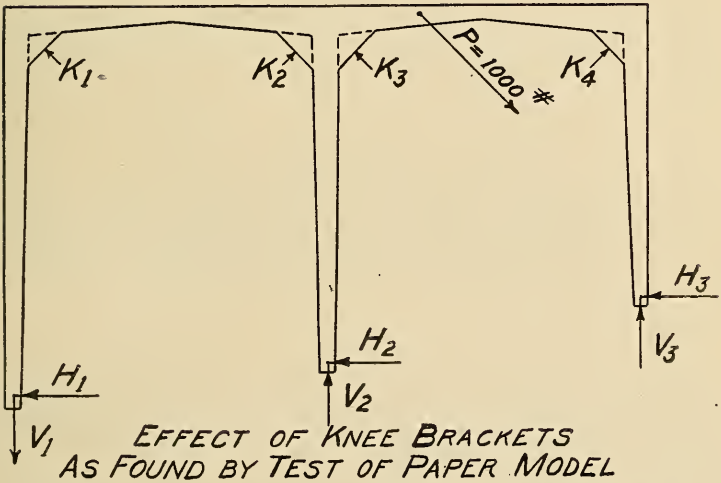
The structure in Fig. 23 presents no more difficulty than the preceding one when solved mechanically. This problem was completely solved by reading the twelve deflections at point *E* corresponding to the *H*, *V*, and *M* displacements introduced successively by the four gages at fixed sup-



ALL VALUES OBTAINED WITH PAPER MODEL

	A,B,C,D Fixed			A,B-Fixed; C,D-Hinged			A,B,C,D-Hinged		
	H	V	M	H	V	M	H	V	M
A	39 →	19 ↑	193 ↻	25 →	9 ↑	146 ↻	10 →	6 ↑	0
B	167 →	481 ↑	535 ↻	156 →	477 ↑	477 ↻	95 →	482 ↑	0
C	173 ←	493 ↑	415 ↻	144 ←	477 ↑	0	108 ←	550 ↑	0
D	34 ←	5 ↑	142 ↻	34 ←	26 ↑	0	7 ←	24 ↓	0
Ck.	$\Sigma H = 1$	$\Sigma V = 2$	$M_E = 121$	$\Sigma H = 3$	$\Sigma V = 13$	$M_E = 13$	$\Sigma H = 10$	$\Sigma V = 14$	$M_E = 100$

FIG. 23.



EFFECT OF KNEE BRACKETS AS FOUND BY TEST OF PAPER MODEL

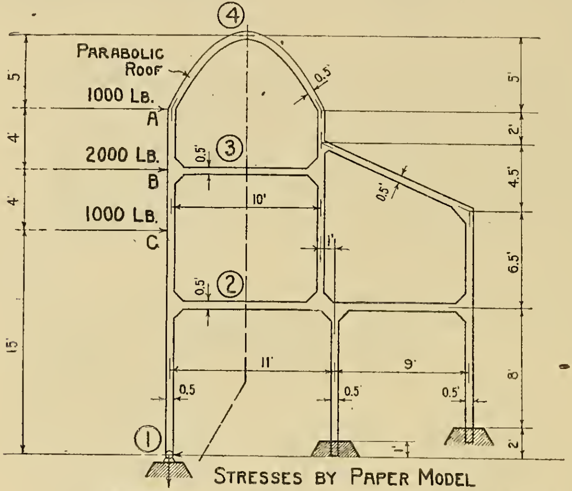
BRACKET OUT	H_1	H_2	H_3	$H_1+H_2+H_3$
NONE	137	222	337	696
K_1, K_2	116	213	370	699
K_1, K_2, K_3	107	220	366	693
K_1, K_2, K_3, K_4	140	221	330	691

FIG. 24.

ports *A*, *B*, *C*, *D*. The effect of changing the supports from fixed to hinged was easily investigated after removing clamping bar from the gage (Fig. 19) and allowing the paper to turn freely on the hinge pin. The values of reactions obtained by deflections are seen to satisfy the three laws of equilibrium very closely.

The structure shown in Fig. 24 was tested by deflections to determine the effect of knee brackets on the reactions. Note the great change in H_1 and H_3 as the four brackets are removed in succession. Any theory that omits the effect of knee brackets evidently may lead to large errors.

In order to show the generality of the model method, the reactions



STRESSES BY PAPER MODEL

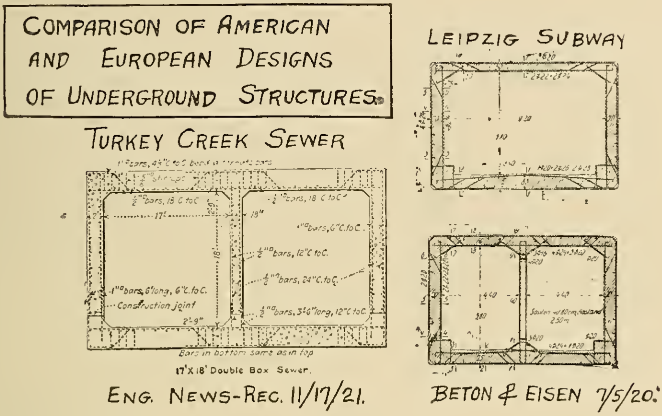
PT.	1	2	3	4	CHECKS
H	139 ←	1145 ←	2332 ←	380 ←	ΣH = 4 →
V	2080 ↓	1145 ↓↑	654 ↓↑	216 ↓↑	ΣV = 63 ↓
M	0	277 ↻	184 ↻	688 ↻	$\Sigma M_B = 322$ ↻

$$\frac{\Sigma M_B}{4000} = \frac{322}{4000} = .08'$$

FIG. 25.

and internal stresses in the complex elastic structure in Fig. 25 were determined by model deflections and the results checked by the three laws of equilibrium applied to the portion of the structure to the left of the dotted section. Considering the magnitude of the applied loads, 4000 lb. total, the check by statics is very satisfactory.

In Fig. 26 attention is called to two designs of similar concrete structures, one made in America and the other made in Leipzig. The assumption of partial continuity is made in the American design, while complete continuity is assumed in the European design. In the American design (Fig. 27a) the top and bottom beams are assumed continuous and the three columns are figured as hinged. This cuts the structure into five distinct structural elements. In the European design Fig. 27b, the complete continuity assumed necessarily leads to a stiffer structure. Fig. 27c and Fig. 27d indicate the manner in which the two structures deflect under load.


COMPARISON OF AMERICAN AND EUROPEAN DESIGNS OF UNDERGROUND STRUCTURES.

TURKEY CREEK SEWER

17' X 18' Double Box Sewer.

ENG. NEWS-REC. 11/17/21.

LEIPZIG SUBWAY

BETON & EISEN 7/5/20.

FIG. 26.

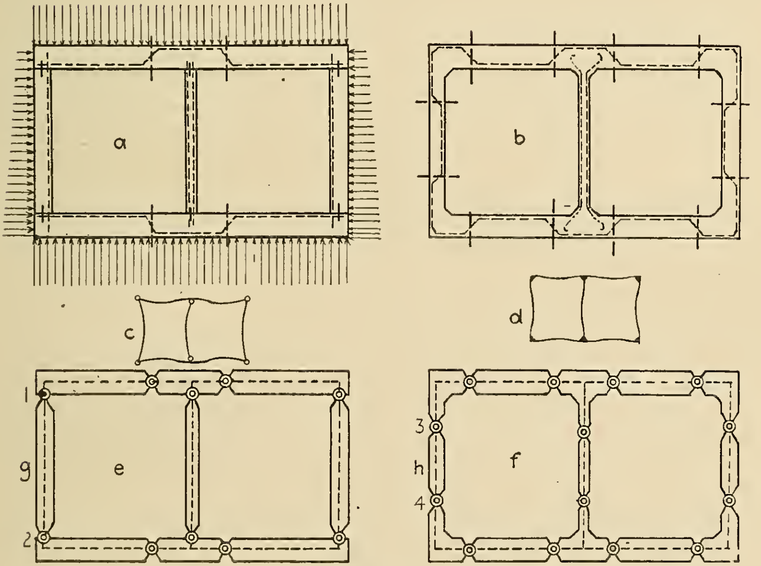


FIG. 27.

Evidently the American design is either structurally weak or structurally uneconomical as compared with the European design. Consider the bending of the side walls *g* and *h* in Figs. 27e and f. In Fig. 27e the side wall has an effective span 1-2, whereas in Fig. 27f the effective span is the distance 3-4 between the points of contraflexure. When it is remembered that the bending moment due to distributed load varies as the square of the effective span length, it is apparent that there is much greater strength and economy in the structure designed and built as a fully continuous structure. One reason why American engineers have not recognized or planned for greater continuity in their designing is because of the laborious mathematical work involved. If, however, the analysis by elastic models is demonstrated to be a practical method of solving the more difficult problems in concrete structures, and further if the designer may be assured of the strength and uniformity of the concrete, the hope may be expressed that in the near future the designer will be more ready to mold the most adaptable of materials, concrete, into a greater variety of stiff, strong, and economical structural forms than have yet been conceived or attempted.

DISCUSSION.

Mr. Lord. A. R. Lord.—I would like to ask Prof. Beggs whether or not he has thought very much about the application of this in reinforced concrete design? As I have listened to the paper, it seems to me that the paper model may represent the actual thickness of the concrete, but it is not possible to represent the effect of the reinforcing steel on the moment of inertia. How would you take account of the very large difference in the moment of inertia which is effected by the reinforcing steel, but does not change the exterior dimensions of the member?

Mr. Beggs. G. E. Beggs.—I have thought of that. In the design of an indeterminate concrete structure, the proportion between the moments of inertia of the various members is first assumed. By means of a paper model of this structure, the distribution of stresses is then determined by methods already described, and the corresponding sections and reinforcement of the actual structure designed. A second model may then be made, if the refinement is desired, with the width of its members proportional to the cube root of inertia $(I + n \cdot I)$ of the actual section used. This second model is then tested to determine the new distribution of stresses, and the sections and reinforcement of the first design modified accordingly. It is seldom necessary to revise the first estimate of stresses more than once, for it is only the proportion and not the actual section of the members that determines the distribution of stresses. Considerable changes, too, can be made in the proportion of members with small change in the distribution of stress.

A Member. A Member.—I would like to ask Prof. Beggs if there is any necessary relation between the elasticity of paper and celluloid and concrete. That is, if you should take wood or some other material for your model, would it make any difference?

Mr. Beggs. Prof. Beggs.—There is no reason why paper, celluloid, or any other material should be selected for the models, except that the material used should have a constant modulus of elasticity within the very small range of stress produced by the small deformations arbitrarily applied to the model. Those familiar with the calculation of arches, tunnel sections, and other indeterminate structures, know that the value of the coefficient of elasticity cancels out in the solution for any unknown stress. The results for the stresses are independent of the coefficient of elasticity of the material, as long as you assume that coefficient constant. I claim that the solution which I get with a model made of a material with a coefficient of elasticity that is constant within the range of stress developed, is in agreement with the present requirements of the Institute, where you specify that the "modulus of elasticity of concrete is constant within the limits of working stresses." If you change that specification, I will need to change my method and you will be obliged to change your formulas.

Mr. Richart.

F. E. RICHART.—The mechanical method of analyzing structures presented in this paper is both unique and simple in application; it seems to be of especial value because statically indeterminate structures in general hold many possibilities for economy of material, and any method which will enable us to obtain a better idea of such structures is worth while. This method, in showing the actual deformations of a miniature structure, appeals directly to the physical sense and enables one to visualize the action of the structure. Regarding the other methods of treating such structures, I might say that a purely analytical method may have available the same method of verifying results; for example, the algebraic application commonly made of Maxwell's theorem may consist in calculating the same deflections that are measured in the tests of models, with the equations of static equilibrium held in reserve for purposes of checking results.

Regarding the model in which knee-braces or brackets were used, I think there is at present very little information at hand on the effect of such changes in cross-section of the members of a frame. Such variations are rather difficult to treat by analytical methods. Several years ago, tests of large reinforced concrete rigid frames were made by the Emergency Fleet Corporation and reported by Mr. Slater to the American Concrete Institute in 1919. These frames had brackets somewhat similar to the model described by Prof. Beggs, and I think it is worth emphasizing that while the brackets may not produce reactions greatly different from those in similar frames without brackets, there will be a large difference in the distribution of bending moments in the two cases. The material used in brackets is placed where it is most needed and is therefore most effective; the result is usually to decrease the bending moments elsewhere, thus making possible a considerable economy in material. In the tests of indeterminate frames cited the loads and thrusts were applied and measured by means of hydraulic jacks and gages. The ratio of these applied forces at the various increments of load throughout the tests showed a fairly good agreement with theory, except at the point of failure when the elasticity of the structure was greatly decreased. That is, the tests of the reinforced concrete frames showed similar elastic behavior to that shown by the paper models.

Besides its use as a classroom demonstration, the method of analyzing structures by the use of paper models apparently should have two distinct fields of usefulness. In one it would be very useful as a rough check which any designer could make on the calculations for his structure; in the second it would be used in a more scientific way so that one who had the microscope, the special gages and all the necessary apparatus to do very fine work could make a study of such problems as the effect of fillets, knee-braces, haunches or other conditions producing a variable moment of inertia in the members of a continuous strutcure.

Mr. Westergaard.

H. M. WESTERGAARD.—In the usual analysis of multiple indeterminate structures one forms and solves sets of simultaneous linear equations.

Several machines have been devised in the past by which sets of linear equations may be solved mechanically; these machines might be expected to reduce considerably the work of computation in structural analysis, but none of them seem to have worked satisfactorily. Prof. Beggs' models and gages, on the other hand, perform the double task of forming and solving the equations applying to complex frames. The accuracy seems to be satisfactory; in fact, the paper models appear to follow the rules of statics and elasticity more closely than do the concrete structures which they represent. The method will be welcomed by the student of structures, who by it may increase his general experience and improve his structural sense; structural sense matters in design as much as does the ability to perform computations. One may expect that the method will be found very useful in the investigation of complex frames. **Mr. Westergaard.**

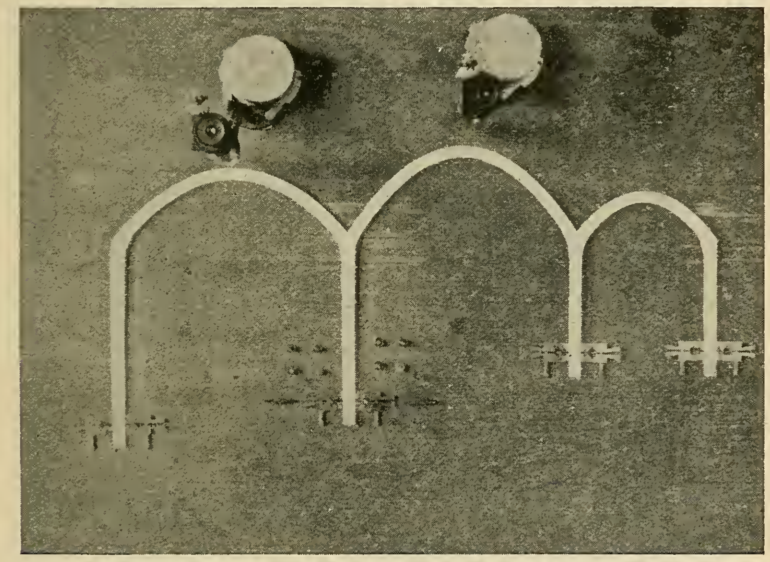


FIG. 28.

Mr. Beggs. Prof. Beggs.—Much interest has been expressed by the members of the Institute in the method of testing the models and in speculations as to the practical application of the method to concrete frames of highly indeterminate forms. In order that the reader may have a clear understanding how the instruments appear in use, there are reproduced in Figs. 28 and 29 photographs taken of the tests for which the numerical results are recorded in Figs. 23 and 25 of the foregoing paper. Small steel balls support the models free of the drawing board, which is in a

Mr. Beggs. horizontal position. The gages at the intermediate sections in Fig. 29 are seen supported on a frictionless bearing made of two glass plates with steel balls between them.

Remarkable concrete structures of indeterminate forms are commonly used in Europe both for utilitarian and esthetic purposes. If the American engineer will perfect the mechanical method of analysis and so avoid the tedious mathematical calculations now incident to the design of these structures, we may hope to see in our own country concrete buildings of greater economy and more attractive simplicity.

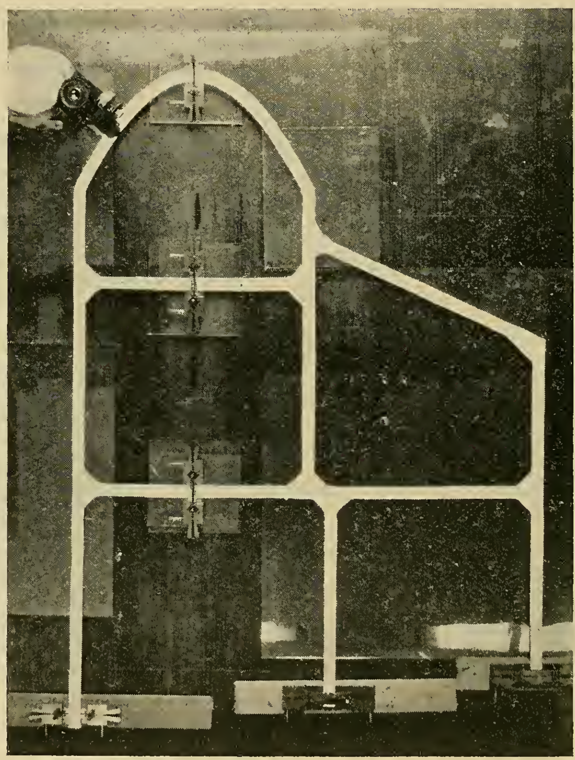


FIG. 29.

CONCRETE SHIPS CONSTRUCTED BY U. S. SHIPPING BOARD.

By Walter R. Harper.*

Among the many new forms of construction that were developed and made necessary by the great demands of the war preparations for construction of all kinds probably none was of more interest to the members of this Institute than the construction of concrete ships by the U. S. Shipping Board during this period.

When it became evident that even by using the entire production and resources of the country, that materials for the construction of steel, and wood ships, to replace those being destroyed by submarines could not be furnished in sufficient quantities, or rapidly enough to make the replacement an effective check to the damage done to shipping by submarines. Other methods for ship construction were suggested, and investigated. This resulted in the decision to build concrete ships. A few reports have been made of the methods used in this new and very interesting form of concrete construction, but scarcely any mention has been made of the completion and successful operation of the ships built.

Little is known about these ships by the public in general, owing to the fact that they were tied up at various ports soon after completion, along with all other shipping when the depression in the shipping business made it necessary to tie up over 50% of the ships in the country. Many people are not aware that these ships were successfully completed and placed in operation and that they performed every function for which they were designed, proving the practicability of this type of construction for ships. When the Shipping Board decided to build concrete ships steps were immediately taken to gather all data on the subject. However, it was found that very little authentic data existed as nowhere in the work had engineers heretofore attempted to design or build concrete ships of a size sufficient for ocean navigation. Numerous small boats or barges for use on rivers and in harbors had been built and operated with success in several foreign countries,—Norway, Sweden, France, Germany, Holland, Spain and a few in Canada and in the United States, but these were all small and of the barge type, the largest being not over 800 tons, and most of them much smaller.

It can readily be seen that the design for a 7500 fon ship to withstand the stresses due to storms and other conditions of ocean navigation would vary greatly from that of the small boats heretofore built abroad. Many problems arose which had not been heretofore considered, making this work pioneer work in this type of shipbuilding. The Shipping Board began numerous tests, experiments, and investigations of materials and methods to be used in the construction of concrete ships. Tests were made of concrete composed of various aggregates, of different mixtures, of different types of reinforcing, and different methods of fabricating. Numerous slabs and beams were built up and broken in tests for strength. Concrete mixing machinery of all makes was tried out and the mixtures produced tested in order that only machines producing concrete of a maximum strength should be used.

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* Concrete Engineer, U. S. Shipping Board.

The first tests and experiments were made at the laboratories of the Bureau of Standards at Washington. The later experiments were conducted in the University of Pennsylvania laboratories at Philadelphia. Observations were made on ships actually in operation at sea in all kinds of weather, and under various conditions to ascertain the forces which would cause stress in the members of a ship's structure. These forces were measured by attaching strainograph instruments to members under stress. Up to this time there had apparently been no actual measurements taken of these forces, values for them had been assumed. It can readily be seen that an economical design for a concrete ship could not be made until the forces acting on the structure were known. The necessity of reducing the weight of the ship to a minimum figure and still retaining a maximum strength in all members, made it imperative that no excess material be used.

It was realized at first by the engineers of the concrete ship section of the Shipping Board that a concrete ship would be handicapped by excess weight if the concrete was composed of sand and gravel or broken stone. Investigations were started of other aggregates and experiments made with lighter materials. It had been decided that a concrete with a compressive strength of at least 4000 lb. per sq. in. after twenty-eight days should be required. Various light materials were used for aggregates during these investigations, such as slag, broken clay, tiles, hollow clay granules, cinders, volcanic rock, etc., but none gave the desired results. Some gave lightness but were lacking in strength. Slag gave a concrete much lighter than sand and gravel but contained chemicals which were detrimental to the reinforcing steel. The volcanic rock used as an aggregate gave a concrete of the lightness and strength desired but the deposits were so far removed from the proposed shipyards that the question of transportation made its use prohibitive.

After many experiments it was decided to imitate the volcanic rock by baking a clay into a clinker in a kiln then crushing and screening the result for the aggregate. A clay containing vegetable matter was selected. This was baked in a kiln, at a temperature of about 2000 deg. F., the vegetable matter turned to gas inflating the clinker leaving disconnected isolated cells when cool. This material was screened after crushing in a stone crusher and well graded material passing through a half inch screen and held on a quarter inch screen was used for the coarse aggregate. Well graded material from ¼-in. size to fine was used for the fine aggregate. This used as an aggregate gave a concrete weighing 105 to 110 lbs. per cu. ft., compared with 150 lbs. per cu. ft. when sand and gravel was used for an aggregate, saving nearly one-third the weight of the concrete by the use of this light aggregate. Crushing strength of this material when made up into 1 : 1 : 1 concrete cylinders 6 in. in diameter and 12 in. in height after 28 days varied from 5000 to 6000 lb. per sq. in. By the use of this concrete a saving of approximately 1000 tons was made in the weight of the 7500-ton concrete tankers built, meaning that a thousand tons more cargo could be carried by the ship. From the results of the carefully conducted investigations, and tests, methods were selected which were to be used in the construction of the concrete ships and specifications were written. However, as no previous work of this kind had been done conditions often arose in the field which made it necessary for the engineer in charge to change methods previously decided upon and substituted others.

Designs for three different types of ships were made. First a cargo carrying vessel of 3500 tons d.w. and modelled along the lines of the 3500 tons d.w. Ferris type wood ship,—282 ft. in length and 46 ft. beam. These vessels were coal burners with 1400 hp. engines located amidships, using the same machinery that was used in the 3500 ton wood ships built by the Emergency Fleet Corporation. They were designed for 10½ knots speed. Two ships of this type were built at Wilmington, N. C., the "Cape Fear" and the SS "Sapona." The second design was for an oil tanker of 7500 tons d.w., 434 ft. 3 in. in length, 54 ft. beam. This type was outfitted with oil burners with 2800 hp. engines located amidships and designed for a speed of 10½ knots. Eight ships were built on this design, two at Jacksonville, Fla., two at Mobile, Ala., two at San Diego, Cal., and two at San Francisco, Cal. The third design was for a 7500-ton cargo boat. Construction work on two of these was started, one at Mobile, and one at San Francisco. These ships were not completed owing to an order received by the Shipping Board soon after the signing of the armistice to discontinue all new ship construction.

In addition to the above standard designs two other concrete cargo vessels were built by the Shipping Board. The "Atlantus" 3000 tons d.w. built at Brunswick, Georgia, at the Liberty Shipbuilding Company's Yard, and the "Polias" 3500 tons d.w. built at the Fougner Yard at Flushing Bay, Long Island, N. Y.

The first work of construction was done on the "Atlantus" at Brunswick, Ga., in the Spring of 1918. The work on the "Polias" was started soon after the "Atlantus." These two ships were well along towards completion before work was started on the remainder of the concrete ships on the construction program and were used somewhat as experimental stations. New methods were tried out and when found satisfactory the same methods were used later on the other ships being built. The difficulty of executing this work was added to by the fact that no work of this kind had ever been done before, therefore, mechanics, foremen and Superintendents were without previous experience. None of them had even seen a concrete ship as these were the first ocean going concrete ships in the world, except the "Faith" which was built at about the same time, consequently all portions of the work usually left to foremen had to be supervised by engineers, who themselves had only theoretical knowledge of the procedure to be followed. Men who had had experience in reinforced-concrete building and bridges and other concrete structures could be obtained but the details of this work differed greatly from any on which they had been previously employed, greater accuracy being required.

To avoid the danger of the possibility of the corrosive action of salt water on the reinforced steel all rods had to be so placed that they would have a covering of at least ¼ in. of concrete when the forms were removed. The rods which had been previously carefully bent to the exact curves required, from templates, had to be placed with extreme care in order that they would follow the curve of the outside forms and remain ¼-in. distant therefrom and still be retained in the limited space of the form allowed for the member. It can readily be seen that an error in the bending of the rod or placing would bring the steel in contact with the forms leaving it exposed when the forms were removed, or the member increased in size and weight added to the ship by the necessity of adding excess concrete to cover the rods incorrectly placed. Another feature which added to the difficulty of placing the rods in the frames (ribs) and other upright members, was the fact that none of these upright rods were vertical on the ships which were to be launched by stern launching. All upright members which were to be vertical after the ship was launched, such as, frames, bulkheads, etc., were built at right angles to the keelsons, and axis of the ship, and as the axis of the ship was parallel to the ways which had an inclination from the horizontal of ⅝ in. to a foot, it follows that these upright members would have the same angle with the vertical, namely ⅝ in. to a foot, and that all rods and forms would have to be placed at this angle during construction. No difficulty was experienced in placing the forms as the carpenters were accustomed to installing work at any angle by means of a straight edge and plumb line, but to mechanics placing reinforcing steel such accuracy was something to which they had not been accustomed and some difficulty was at first experienced in impressing upon them the necessity of the extreme care required in this work.

The first ships built were for stern launching. Those built later and all of the oil tankers were constructed for side launching which simplified the work of placing reinforcing steel and forms. Owing to the side launching ways, being laid horizontal, and as the axis of the ship was parallel to ways, all members of the ship's structure were horizontal or vertical. In order that concrete work might be carried on throughout the year climatic conditions were taken into consideration in selecting cities for the yards in which these ships were to be built. Two yards were located on the Pacific Coast, the remainder situated in the South with the exception of one located on Long Island, N. Y. That work could be more economically done at a location in a mild climate was demonstrated at the one yard situated in the North where we were subjected to an extra expense for housing in a portion of the work during the winter and establishing a temporary heating plant to prevent freezing of completed work, and were also obliged to heat aggregates and water used in the concrete.

It is the opinion of the writer that concrete ships can be more economically built on the Pacific Coast where we have neither extremes of heat or cold. The work on all the ships was similar with only light variations. A description is given here of the methods followed in the construction of the "Polias" built at Flushing Bay, Long Island, N. Y.

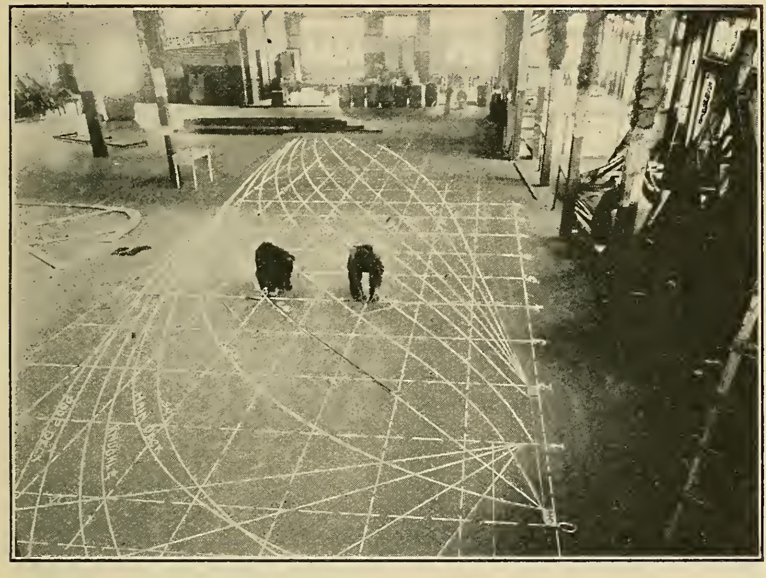

FIG. 1.—LAYING OUT THE WATER-LINE PLAN FOR THE "POLIAS."

MOLD LOFT WORK.

The first work done was to prepare the ways on which the ship was to be built. While the ways were being constructed, work was in progress in the mold loft and carpenter shop. The mold loft work was similar to that in a shipyard for steel or wood ships. Water lines and a body plan were laid down full size enlarged from a quarter-inch scale drawing, then faired up to true curve. These two plans were simply horizontal and vertical projections of the ship taken at the intervals mentioned, the body plan being the vertical projection at the frames (ribs), and the water line plan being the horizontal projection of the outer surface of the ship at its intersection with the water, taken at 3 ft. intervals, in this case. The mold loft was laid off in 4 ft. cross sections corresponding to the frames (ribs) of the ship, which were 4 ft. on centers. Lines were laid down for the 1 ft. 6 in. water line, the 3 ft. water line, and then at intervals of 3 ft. until the deck was reached.

These were faired up and transferred to the body plan, which was laid off with sections through the ship for each 4 ft. at the frames (ribs), as usual, only one-half of the plan being necessary for water lines, the aft, and fore body plans. The port side was used for this. From the lines of the body plan, templates were taken off at each frame (rib), following the outside lines from the center line to the deck, allowance being made for the ⅞ in. thickness of the sheathing for the outside forms. These templates were taken to the carpenter shop, fitted to bents previously made up for each frame from detail plans, and the lines of the ship marked off with a pencil from the template on the outer board of the bents, which was then cut to the curve, making the form complete (see Fig. 3) and ready to set up on the ship. The completed bents were taken to the ship and carefully set up to the proper line and angle by the engineers. (The frames of a ship are sometimes called the ribs.)

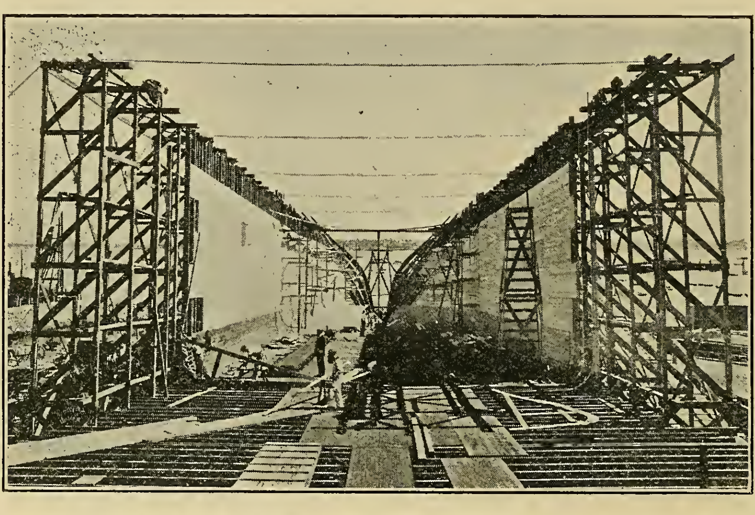

FIG. 2.—BUILDING FORMS FOR SIDES AND BOTTOM.

Note 16-Ft. Panels and Double Stringers for Removing Panels and Bottoms.

The deck line and center line, which were marked on the bents, were used for adjusting the bents for line and grade when setting. All bents had the same angle, from a vertical line, as the inclination of the ways from a horizontal line (⅝ in. to a foot). Templates for the inside forms were taken from the body plan on stiff mold loft paper, allowance being made for 5 in. thickness of the shell. The inside forms for the bottom frames, keelsons and bilge curve were made by adjusting boxes, boxes being made to accurate sizes, from these paper templates. One and one-quarter inch spruce was used for the inside forms. Templates for the location of all inserts or anchor bolts were also made in the mold loft from full sized drawings on the mold loft floor showing the exact location of the machine or portion of the outfitting to be attached by the insert. These had to be very accurately located, as they could not be moved after concreting. There are approximately 5000 inserts in the boat, being the bolts for anchoring machinery, mast tables, deck stanchions, pads, deck rails, shrouds, etc.

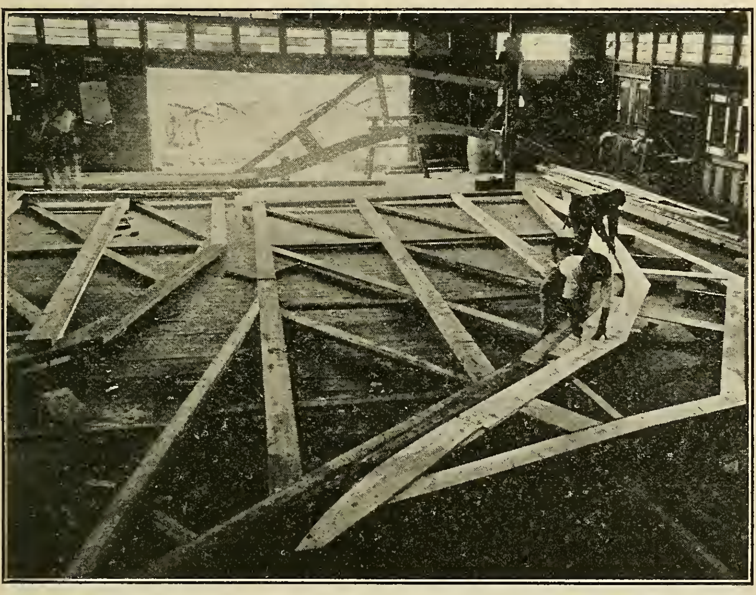

FIG. 3.—TEMPLATES USED IN MARKING LINES ON BENTS IN CARPENTER SHOP.

On the completion of the ways, work was immediately started on blocking up for the ship. Seven lines of longitudinal stringers were blocked up from the ways, one under the center keelson and under each side keelson, and four lines of 6 x 6 in. stringers supported on 6 x 6 in. posts, space being left between the posts to insert the sliding ways after the bottom forms were stripped, this gave longitudinal supports every 6 ft. for the bottom forms. On these longitudinal supports, transverse stringers 2 x 6 inches were placed 2 ft. on centers, except under bulkheads, where they were placed 1 ft. on center. These stringers were inclined to the center with a 6 in. drop to conform with the dead rise of the ship. The stringers and posts were doubled at intervals of 16 ft., so that they could be removed for a space of 16 ft. in width across the bottom of the ship, when stripping forms, painting, and reblocking.

Levels were taken daily on bench marks located on the forms, and the wedges under the blocking driven up when any settlement was indicated. A very slight settlement would have caused cracking in the green concrete. Any settlement of the foundations of the ways from overloading was taken care of in their design, also any slight crushing of the timbers of the supports or forms was taken care of in the design. However, settlement due to shrinking of the timbers in the stringers and other supports could not be prevented. This was reduced to a minimum by endeavoring to have as few horizontal timbers for supports as possible, using vertical timbers or posts when practicable, which of course have less shrinkage than horizontal timbers. By constant watch on the levels and wedges all settlement was prevented. The supports and blocking under the ship had to be set and carefully replaced three times during the construction of the ship. First, when removing the bottom forms this was done by removing the supports and forms for a space of 16 ft. wide across the entire bottom of the ship. The panels of the forms had been made in 16 ft. lengths to allow for this removal. The blocking and posts were then replaced in the 16 ft. space and wedged up to the new concrete forming the bottom of the ship. The next 16 ft. space was then treated in a similar manner and so on until the entire ship had been gone over and new blocking set ⅞ in. higher to fill in the space previously occupied by the bottom forms. A similar operation was performed when painting the bottom and still another when the blocking was finally removed and the sliding ways for launching placed under the ship. During these operations settlement had to be constantly guarded against. The above description illustrates the extreme care necessary throughout the construction of the ship.

FORMWORK.

On Aug. 20, 1918, the first work was done on the ship, when the scaffolding for the outside forms was erected at the midship sections. The bents for the scaffolding at midships were rectangular and made up in the carpenter shop from details and accurately placed in position on the ways, being bolted to holes left in the concrete beams of the ways. Fillers cut to the proper curve of the bilge, which was a 3 ft. radius, were then fitted in tangent to the bottom stringers at one end and tangent to the inside of the scaffolding at the other, thus making a support for the outside forms, which had the mold of the ship at this section. The scaffolding, which formed a mold for the outside forms at the curved sections of the boat, fore and aft, had pieces cut to the accurate curve of the boat at each frame, which were fitted on to bents in the carpenter shop, from templates made in the mold loft, as previously explained. The sheathing for the outside forms was carried along as the bents were erected, which was composed of ⅞-in. T. & G. flooring 3 in. wide. The sheathing was made up into panels 16 ft. long and 3 ft. wide for the bottom, and straight sides for midship section. The sheathing for the curved portions, fore and aft, could not be made up of panels and was fitted in piece by piece to conform with the lines of the ship. The sheathing for the bottom and bilge curves was so placed that the concrete was molded to the lines of the ship when stripped, but just above the bilge curve at the 6 ft. water line, the outside forms were made to project in ½-inch making an offset in the concrete when the forms were removed. This space was filled in with mortar from a cement gun and brought flush with the outside of the ship at the bilge and given a float finish with a metal trowel, thus doing away with disfiguring board marks for the forms. The "Polias" was the only ship treated with this stucco finish. The results were satisfactory.

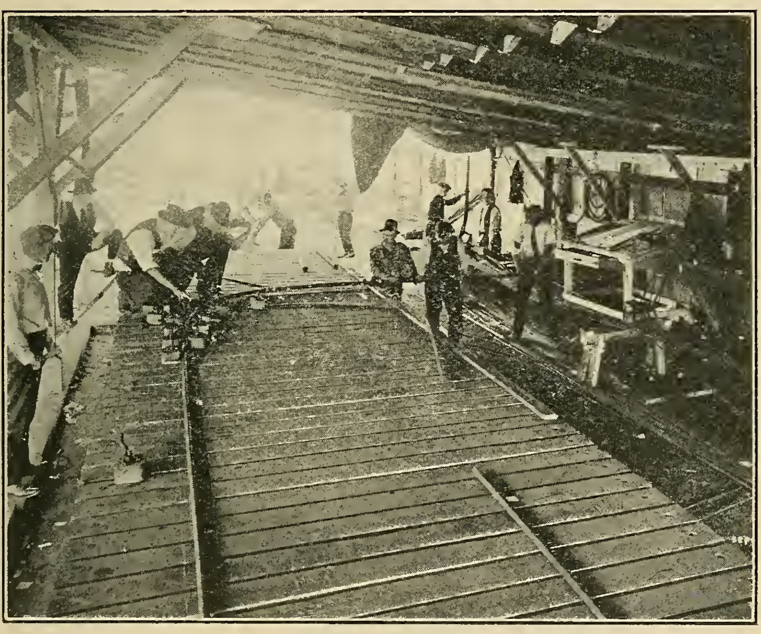

FIG. 4.—BENDING STEEL FOR FRAMES OR RIBS.

STEELWORK.

The work of bending steel was started on Aug. 30, 1918. The bottom rods were 60 ft. in length and designed with a 4 in. hook on each end. The bending of these hooks proved to be very satisfactory with Koehring benders, with cogs and ratchet attachments. When the work was started, there were only two benders on the work, one Koehring bender and the other an ordinary lever. Out of the first forty hooks bent, six were broken with the lever bender, while the Koehring bender, working on the other end of the same rod, had not broken any. All of the steel was high carbon intermediate steel, being high in tensile strength but rather brittle and springy and extremely hard to bend to the exact shapes. The bottom rods 1¼ in. in diameter were the first steel to be installed in the boat. They were laid 4 in. on center and lapped 30 diameters, with a hook on each end.

No attempt was made to wire the laps, but they were kept one inch apart, allowing the concrete to flow between the laps. The rods on the bottom, and up to the bilge, were placed on precast cement blocks, 4 in. square and 1 in. thick, with a groove in the edge to give the blocks additional bond with the concrete. The blocks on the side, up to the bilge, were cast with wire embedded in them, which was lapped around the rod to hold the block in place and the rod away from the outside forms. These blocks prevented the steel from resting on the forms and allowed the concrete to flow under the rods. The shell steel was composed of ½ in. and ⅝ in. square deformed bars laid diagonally, 4 in. on centers. There were four layers of this diagonal steel in the shell. The two outside layers extended from the bottom up and interlaced with a similar steel of the bulwarks. The two inside layers extend across the bottom of the

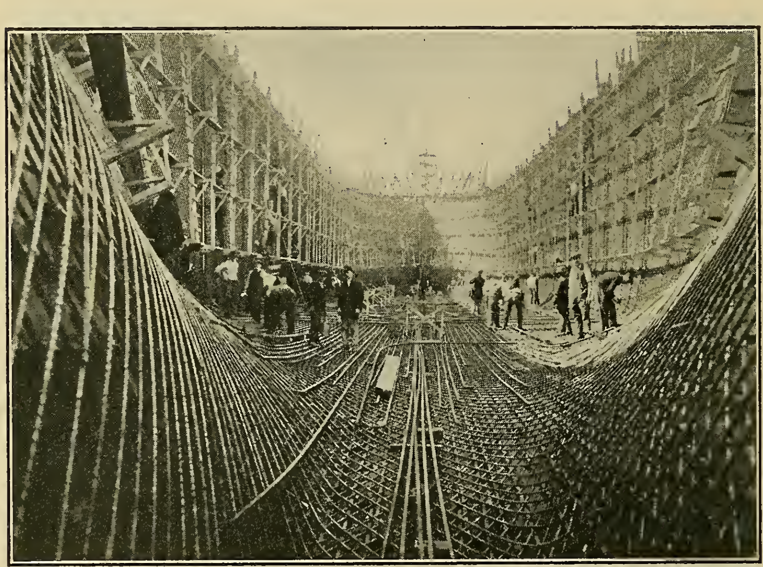

FIG. 5.—SHOWING SHELL STEEL ON BOTTOM, LONG RODS IN CENTER ARE FIRST RODS OF KEELSON.

ship, up the inside of the shell and bent over and were interwoven with the deck reinforcing steel.

The inner and outer diagonal shell steel was separated by a 1⅛ in. rod, being the first rod of the frames. The outside diagonal shell steel differed from that of the bottom, in that instead of being supported on precast blocks, it was nailed directly to the forms, above the 6 ft. water line, where the forms projected in one-half inch, as before mentioned. When removed, the steel was exposed in the concrete, it was then covered and protected with a half inch coat of gunite. The "Polias" was the only ship built with diagonal shell steel. The other ships were designed with the shell steel parallel with and at right angles to the axis of the ship. The steel for the frames was composed of 1⅛ in. deformed square bars, stirrups for the same having been made from ⅜ in. deformed square bars. It was necessary to bend the 1½ in. bars very accurately, in order to get the required number in the assembled frame or rib and not to exceed the space allowed for the same, as any increase in the size of the frame would necessarily increase the weight of the boat. The bending for these bars was done on a large bending table, made of oak planks, bolted to the frame of the table at a convenient height for working.

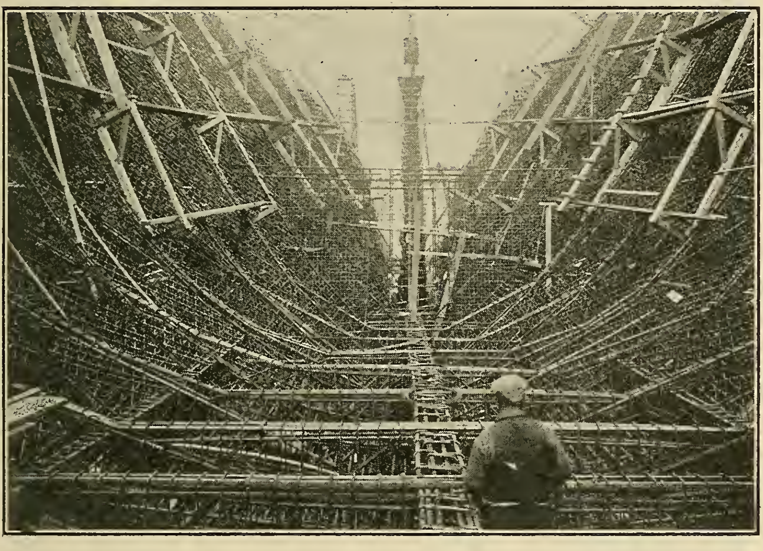

FIG. 6.—LOOKING AFT SHOWING STEELWORK AND FRAME STERN FRAME PARTLY SET.

All Steel Was Accurately Placed by Steel Tape Measurement.

A space of ½ in. was left between each plank, in which were fitted dogs with a thread and clamp running in the space between the planks. Templates taken from the body plan in the mold loft were laid on the table, and the exact curve of the frame desired marked in chalk. A bar of steel was then laid along the line and the dogs adjusted to it. Two men forced one end of the bar around the curve, which was marked by the inside dogs, then the outside dogs were clamped up to it and the curve continued until the bar had the shape desired. The intermediate steel used contained a very high degree of carbon, making the bars very elastic and difficult to bend to an exact curve. It was necessary for the end to be taken far beyond the chalk mark laid down on the tables then, as the pressure was removed, it would spring back to nearly the right position; if not, it was again bent until it had the exact curve of the frame. After a few days' work on the table, the men became very proficient in estimating the amount of spring the bars had, and very little time was lost by excess bending. In bending these, there was a tendency for the square bars to twist and bend on the diagonal making them occupy more space in the frames than was intended.

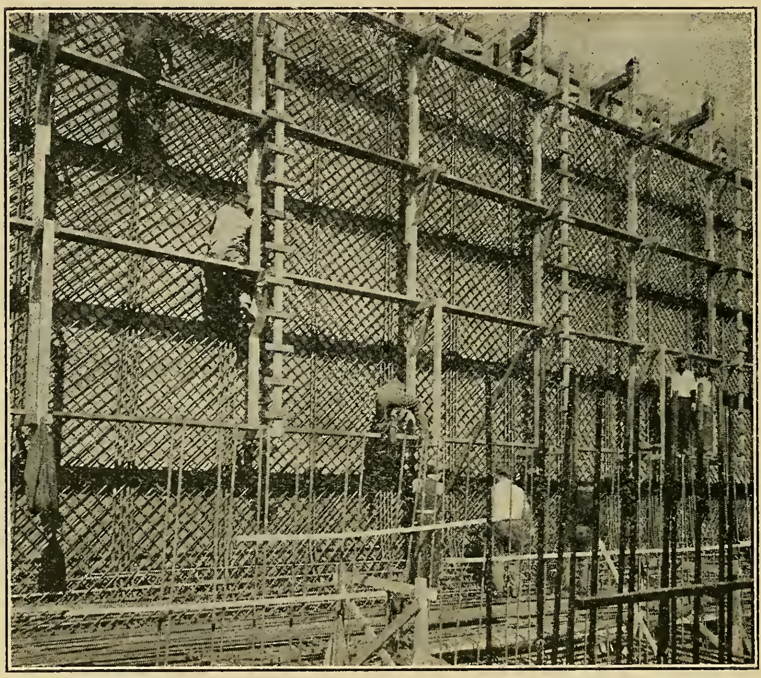

FIG. 7.—DETAILS SHOWING SHELL STEEL, FRAME STEEL BETWEEN SHELL, AND FIRST STUD FOR SIDE KEELSON.

It would recommend in future work that plain round bars be used, also a degree of steel with less carbon (structural grade only). It was impossible to obtain this grade of steel during the construction of the boat, owing to war conditions. Some of the bars for the large beams and keelsons were bent on this table, although the Koehring machine (previously described) was used for the majority of the beam and keelson steel. After bending the steel for all frames, keelsons and girders for hatch coamings, it was tagged and marked before taken to the boat, where it was assembled and placed before the forms were erected. Later on a machine bender operated by compressed air was used instead of hand bending.

The steel for the deck beams, pilasters and for the bulkheads was assembled outside of the boat and placed in the forms already erected. After the steel for the frames, beams and keelsons had been carefully bent on the table and taken to the boat, it was accurately put in place, measurements being taken with steel tape to locate each beam, frame, girder, keelson, etc. None of the steel for the frames of bulkheads could be laid vertical, owing to the fact that they were perpendicular to the keelsons and the bottom of the boat, which naturally had the same inclination as the ways to a horizontal line (⅝ in. to 1 ft). This caused the steel in the frames to have the same angle (from a vertical line) as the inclination of the ways. This added to the difficulty in laying, so that the entire work could not be trusted to ordinary foremen or mechanics, but was supervised by an engineer who was constantly on the work with tape and instruments. All forms also were erected under the direct supervision of the same engineer. This accuracy was carried on throughout the boat in every operation. Many things usually left to foremen and mechanics were conducted directly by engineers. The designer of the steel work was constantly on the work during its erection.

CONCRETE WORK.

The concrete used in the ship was composed of Dexter cement, reground especially for the Shipping Board, 90 percent passing through a 200-mesh sieve. What is known as Steer's gravel was used for the coarse and fine aggregate. The gravel was dredged from the Long Island Sound at Huntington, Long Island, 50 percent of that used had passed through a half inch screen and held on a ¼ in. screen and 50 percent through a ¼ in. screen and held on ⅛ in. screen. This was washed and remixed in the above proportions for the coarse aggregate. A test of this showed 35 percent voids. The sand was dredged at the same time and was washed and screened from the gravel. It was well graded from a quarter inch down to fine.

The "Polias" was the only ship built in which sand and gravel was used for the aggregate, it having been impossible to obtain at that time the light aggregates of which the concrete in other ships was composed.

The amount of water used in batches was carefully measured by a comparatively new method as follows. Cylinders 6 in. in diameter and 12 in. in height, open at both ends, made of waxed cardboard, were filled with concrete taken from the hopper as the batches left the mixer. They were placed vertically, with one open end, on a platform, the concrete then settled by vibrating the outside of the cylinders. The upper end was struck off flush with top of cylinder, and any excess concrete thrown away. Then the cylinders were raised slowly, allowing the concrete to flow out on the platform at the lower end, leaving a cone of concrete, the height of which necessarily depended on the amount of water in the mixture. It was found from experience that a cone from 3 to 4 in. in height gave a concrete that was about as stiff as could be used by the ordinary workman and at the same time being sure of getting it thoroughly settled around the intricate mass of reinforcing steel, with a reasonable amount of inspection. In laboratory work, or work done by skilled concrete engineers, concrete stiff enough to give a cone of 6 to 10 in. in height could be used, but as before stated, laborers could not be trusted to place concrete stiff enough to produce cones higher than 4 in., without leaving voids between the steel. It is a well known fact that dry concrete will develop greater compressive strength than wet concrete, and as the height of these cones depended on the amount of water in the mixture, it naturally follows that the greater the height of the cone, the stronger the concrete. This fact was borne out in later tests. However, a certain amount of the strength has to be sacrificed in the actual work by using a mixture that is fluid enough for the laborers to settle around the reinforcing, instead of spreading over the bars, leaving voids between them. From my observation, a 3 to 4 in. cone gave the best results on the work, providing the forms were vibrated with pneumatic hammers, which will be described later.

The water in the mixer was measured by an automatic device, which would allow only a certain amount of water in the mixer without changing the adjustment. Having determined the height of cone desired, the water measuring device was adjusted until a concrete was obtained which would give this height of cone. This method can be used in a dry or moist climate, or with aggregate saturated with rain water, and by simply obtaining the same height of cone, we can be sure of obtaining concrete of the same consistency in any part of the country, so that comparisons can easily be made of mixtures for tests. The time required to raise the cylinder from the concrete should be the same in all tests to get a true comparison, as the rapidity with which the cylinder is removed from the concrete tends to regulate the height of the concrete cone left. Very good results were obtained by allowing ten seconds for the raising of the cylinder, same having been measured by a watch. As a matter of fact, the records on file of these tests do not give the height of the cone, but the reciprocal; that is, the height of the cone subtracted from the height of the cylinder (12 in. in this case), calling it the drop. I found that this only tended to complicate the records and that even some of the engineers were confused in regard to the height of the cones and the drop, as the reciprocal is called. These heights were taken once an hour and at the same time other cylinders were filled with concrete and set aside to harden for a 28 day compression test on the compression testing machine. In keeping a record of the cylinder broken, the height of the corresponding cone was also kept. Tests showed that there was a fixed relation between the height of the cone and the strength of the cylinders.

This method of measuring the consistency of the concrete is far better for obtaining comparative tests of strength of field concrete mixed in the different parts of the country than the old practice of specifying a certain numbers of gallons of water for each batch or a certain percentage of the weight for water, as the moisture in the aggregate is taken into consideration in this new method. In the field, of course, it is impossible to have the aggregate baked as in laboratory work.

The average compression strength of the cylinders made from concrete used in the bottom pouring was 4900 lb. after 28 days; the average compression strength of cylinders made of concrete used in the shell was 5100 lb. after 28 days. Maximum strength obtained in any cylinder was 6300 lb. per sq. in. after 28 days.

The intricate mass of reinforcing steel in the forms, both in the outside shell and the frames and beams, and in fact the entire ship, made the question of tamping or settling around the steel without leaving voids one of great importance. A very dense concrete, and one thoroughly settled around all of the steel, was obtained by the use of pneumatic hammers or chippers, in which were inserted a small steel bar with the end upset to vibrate on the outside of the wooden forms. The length of the tool or bar used varied from 1 to 3 ft., the long ones being used near the stern and on the deep deck beams. The best results were obtained by vibrating the outside forms of the shell from the inside of the boat. The concrete was poured in the shell through 3 ft. panels from the inside. The hammers then were used to vibrate on the inner surface of the outside forms through the opening for the panel, and also on the inside forms. The bottom concrete was settled by vibrating the bottom forms from the inside about one foot ahead of the fresh concrete. The deep forms that were difficult to reach; such as hatch coamings, up around the stern frame and deck beams, were vibrated by putting the long tool directly in the concrete and vibrating the bottom of the forms, in all cases great care being taken not to vibrate the steel.

When the vibrating was started, it was attempted to vibrate the bottom and shell from both inside and outside, the ship being cross sectioned transversely. The frames were numbered and lettered longitudinally dividing the ship into sections. A man standing on the upper platform, noting where the concrete was being poured and vibrated on the inside, transferred this information to the men vibrating on the outside by a megaphone. However, this was very unsatisfactory as it took several inspectors to keep the outside man informed of the whereabouts of the fresh concrete on the inside, and even at that there were times when the forms were vibrated where the concrete was partially set. This method was in use for only a few hours, when we resorted to the method first explained, by doing all vibrating from the inside, which proved to be just as effective and much more satisfactory. When the outside forms could not be reached from the inside, through the open spaces left for panels holes were bored with augers large enough to admit the vibrating tool, which was pushed through to the inner surface of the outside forms.

The types of pneumatic hammers of the Ingersoll-Rand make were used in this case; the N-1 type of 6¾ lb. using 7 cu. ft. of free air. Experience showed that a larger hammer of about 12 lb. was more useful on the work, as they are powerful enough to run drills for cutting the concrete away, which is necessary in putting in additional inserts. They can also be used for roughing up the joints where new concrete is to adjoin the old concrete. The small hammers were not found powerful enough to do the roughing in a satisfactory manner. Also the larger more powerful hammers were found very convenient in making concrete force its way into congested places. Concrete that was poured into the forms and had apparently taken its level, was lowered from two to three inches by the use of hammers. Concrete was also seen to move at least fifteen feet from where the vibrating was being done on the forms. This vibrating proved very successful, but like all other careful work in connection with concrete ships, it required men of experience to give it the proper inspection.

The mixture used in the concrete was one part Dexter cement, two-thirds parts Steers' sand and one and one-third part Steers' gravel, and enough water to leave a cone of from three to four inches in height. The gravel and sand were carefully measured in boxes which had been built to the exact size required, no wheelbarrow measurement being permitted.

Two ½-yd. Ransome mixers were used on the starboard side of the ship. The mixers were located in pits at the foot of towers. A hopper was placed over the intake of the mixers on level with the ground, so that no platforms were needed to elevate the material before placing in mixers. After the concrete had been turned approximately one minute and a half, it was taken in a skip or elevator up through the center of the towers to the top, where a metal hopper was located, from which it was carried through steel chutes to two hoppers in the inside of the boat, from here it was conveyed to the place where it was to be used, in wheelbarrows or in some instances in buckets. Coal hods were also used and found very successful, the flat nose permitting the concrete to be dumped into the narrow forms without spilling. Small metal hoppers were used in the bottom frames to pour the concrete in to prevent it from falling on the bottom of the boat.

The concrete was conveyed to the shell through small wooden chutes. In pouring the bottom frames and shell, it was endeavored to bring the concrete up to a height of not more than 8 to 10 inches all around the boat, so as not to pour any layer of concrete before the layer below had begun to set, thus making a monolith without construction joints. This object was accomplished except in a few cases. The major portion of the concrete was laid in three operations. The first pouring was on Nov. 5 and 6 for the bottom and up to the center of the longitudinal girders at the 6 ft. water line, just above the bilge, where the joint was made. This consisted of 431 cu. yd. and was laid in 36 hours of continuous pouring, the men working in twelve-hour shifts organized as follows:

Fifteen men loading and transporting cement on flat cars between cement shed and mixers; 13 men on each mixer handling sand, gravel and cement, making a total of 28 men prehandling. The work consisted of loading sand and gravel into measuring boxes, wheeling and dumping into hopper over mixer, and adding cement and water. Two men on each tower at hopper working valve and unloading skip; two men at each interior hopper in bottom of boat working slide gates to load wheelbarrows; 16 men wheeling concrete from each interior hopper. The interior hoppers were arranged with two openings and concrete was discharged in both directions from the hoppers; therefore, the 16 men were divided into 8 men wheeling in each direction, making a total of 32 men wheeling from the two hoppers. Twenty pnuematic hammers were used in vibrating the forms, a man on each hammer; carpenters were used in this case, as it was assumed that the added intelligence of the mechanics would more than compensate for the additional cost over using laborers as hammer operators; 15 laborers unloading wheelbarrows and placing concrete in forms, 1 man employed continually to clean excess concrete dropped on runs from wheelbarrows.

The second pouring on Nov. 25, 1918, was for the shell from the center of the girder, just above the bilge where the first pouring stopped, to a point about 3 ft. below the deck, where the brackets for the deck beams began. This consisted of 365 cu. yd. poured in 31 hours.

The construction joint in the center of the girder was roughened by chippers used in the pneumatic hammers to remove the weak film formed on top of concrete; then gone over with the air hose to remove dust, after which it was washed under pressure and sprinkled with neat cement. I would recommend that the outside forms, for a width of 2 to 3 ft. be made removable along the line of a proposed joint, to allow room to clean and chip the concrete that has been previously poured; also to make an opening through which dust, wire and other rubbish that may fall from above can be removed. A ridge should be left in the center of joint for a key to aid in making it water-tight. To prevent construction joints below the water line, I would recommend continuous pouring of the bottom and shell to the deck. The third pouring for the deck and hatch coamings on Dec. 23 and 24, 1918, consisted of 289 cu. yd. of concrete placed in 14 hours. The decks were intentionally left rather rough, to give a better bond for a bituminous finish to be applied later. The concrete for the poop, forecastle and bridge decks, bulwarks, engine tank tops and shaft tunnel was poured at different times. The aggregates were heated by running perforated steam pipes through the piles of sand and gravel, for all the concrete, except for the first pouring. The water was also heated. This heating had a noticeable effect on the height of the cones obtained to test the amount of water in the mixture. Samples taken at the mixer, where the materials were hot, showed quite a fluid concrete, giving a 4 in. cone. However, the same concrete, after getting cold on the deck, was almost too stiff to place in the forms.

The concrete wall around the top of the engine room was shot with a cement gun. This did not give satisfactory results. The walls sagged before the concrete set, leaving cracks which were patched on the face by the finisher when floating the green surface. When the back panels of the wall (which were used as forms) were removed, after the concrete had set, these same longitudinal cracks or openings, caused by the gunite sagging, were found on the inner side of the walls and had to be patched. Sand pockets were also found. From the results obtained, I could not recommend the use of a cement gun for this class of work, as sand pockets are bound to occur with such an intricate mass of reinforcing. I am also convinced that the gunite costs more for a thin wall than poured concrete, notwithstanding the saving in the cost of the forms.

PAINTING.

All concrete surfaces were given one coat of fluosilicate and two coats of Spar Varnish, applied with a spray under air pressure. The last coat above the water line, on the outside, and on the bulwarks and hatch coamings, contained 55% Spar varnish and 45% white lead by weight, the 45% of white lead having 1% of black carbon, all being ground in a paint mill. The combination of Spar varnish and white lead is said to have three times the elasticity of the plain Spar varnish. It was endeavored to obtain a paint with as much elasticity as possible to prevent cracking from the expansion and contraction of the concrete. The concrete of the first ships was painted inside and outside as described with Spar varnish and white lead. This gave very unsatisfactory results. A short time after painting much of the surface had blistered and peeled making it necessary to remove the paint from the entire outside of the hull by scraping or burning. The hull was then painted with a bituminous paint which adhered to the concrete and gave satisfactory results. The bottom and the portion of the sides which were submerged were painted with anti-fouling paint. The bottom of the "Latham" when drydocked, after it had been in the water three months did not show any evidence of having been painted, the anti-fouling paint having disappeared, leaving the concrete exposed, which was in excellent condition with the exceptions of barnacles adhering to it, the same as would have been found on the bottom of a steel ship. The bottom of the "Latham" was then scraped free from barnacles and a coat of a preparation mixed by the writer applied, which was composed largely of crude oil. It is expected that this paint may prevent the accumulation of barnacles to some extent. The ship has not since been drydocked so an opportunity has not been given to examine the bottom to observe the effect of this experimental coat.

The concrete of all interior surfaces of the ships including bulkheads and interior surfaces of all oil tanks in the tankers were treated as above described with fluosilicate and painted with Spar Varnish. It is the opinion of the writer that this was an expense charged against the ships which might have been saved, as the concrete in the bulkheads and oil tanks proved after using to be absolutely oilproof and waterproof owing to the dense nature of the concrete obtained by using extreme care in the selection of the materials and in the mixing and placing of the same. Undoubtedly the use of pneumatic hammers in vibrating the forms to settle the concrete aided greatly in obtaining this dense oil proof concrete. When portions of the bottom of the "Latham," which also formed the bottom of oil tanks, were cut away during repairs there was found to have been no absorption of the oil by the concrete nor the slightest penetration of oil notwithstanding that these tanks had held oil under a 20 ft. head for three months. This was thick crude oil but we have every reason to believe that similar results would have been obtained with a thinner oil.

For the painting of concrete ships I recommend a bituminous paint for the exterior of the hulls and that only such exterior surfaces be painted as might be required for appearances and for cleanliness, such as portions of the engine, boiler rooms and quarters and that no paint be supplied to the bulkheads and oil tanks. Decks are somewhat improved in appearance by the application of paint, but this is not necessary.

WORK PRIOR TO LAUNCHING ON "POLIAS."

It was necessary to install certain parts of the outfitting before launching; such as the rudder, propeller, tail shaft, stern tube, bitts, mooring rings, hatch covers, hand steering gear, anchor chains, windlass, handrails and stanchions for poor and bridge decks, etc. The remainder of the machinery was installed after launching.

A launching basin 650 ft. in length and 120 ft. in width was dredged to a depth of 22 ft. at hightide, also a turning space, large enough to turn the ship, and a channel to deep water was dredged. The ship was launched with the fore peak and chain locker filled with water to partially trim it while being launched. The ship was classified by Lloyd's Register with New York as the home port. A freeboard of 4½ in. was allowed by Lloyds.

The installation done before launching on the other ships was similar to that done on the "Polias" described here.

FENDERS.

During the operation of the ships it was demonstrated that longitudinal timber fenders added greatly to their protection from shock and abrasion from tugs, docks, piers, etc. I would recommend three rows of longitudinal fenders from the deck line down, composed of 14 x 14 in. oak or yellow pine attached in such a manner that when a fender is carried away during a collision, that the fastening will give away before tearing out a portion of the concrete where it is embedded, causing leaks to develop. Bill irons or protection plates should be installed on the bows to take the abrasion from the anchors, and anchor chains when lowering and raising anchors. The tankers and most of the ships were built without fenders. Had these been included in the original installation many minor repairs would have been unnecessary.

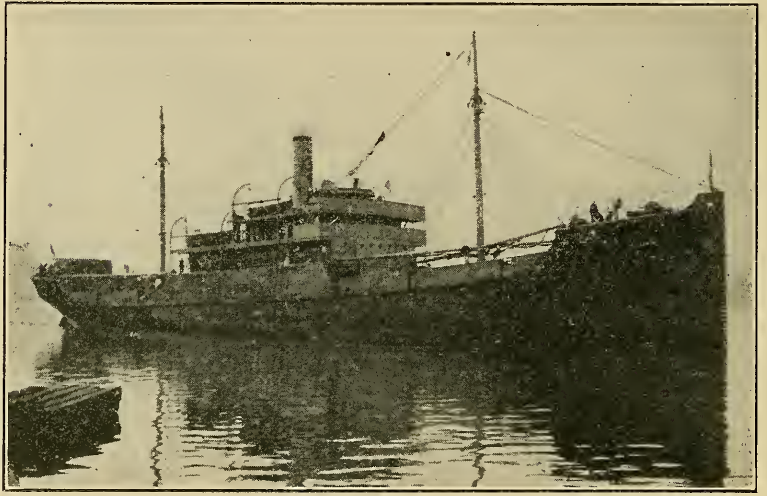

FIG. 8.—STEAMSHIP "POLIAS," 3500-TON DEAD-WEIGHT CONCRETE VESSEL.

VOYAGES AND CARGOES CARRIED BY CONCRETE SHIPS COMPLETED.

The concrete ships completed and placed in commission are as follows:

"*Polias*".—The first concrete ship built by the Shipping Board to be operated and carry a cargo was the "Polias." This was a 3500-ton d.w. cargo carrying ship, length 286 fet. over all, beam 46 feet 6 inches, coal burning 1400 hp. engines,—speed 10½ knots. The hull was built at Fougner Yard, Flushing Bay, Long Island, N. Y., during 1918 and 1919. The outfitting and installation of machinery was done at Providence, R. I., where the hull was towed from Flushing Bay, Long Island. When the machinery had been installed and outfitting completed the ship was given a sea trial and passed the inspection of the U. S. Steamboat Inspectors and Lloyd's Registry. After sea trial the ship proceeded to New York under her own power on Oct. 15, 1919, where it was drydocked. This was her first sea voyage. It was then leased to New York & Porto Rico Steamship Corporation for coastwise service as a cargo carrying vessel between points on the Atlantic Coast, and the West Indies. After drydocking the vessel sailed from New York to Norfolk, Va., empty to load a cargo of coal. It left Norfolk on Dec. 17, 1919, with 2035 tons of coal for Portland, Maine. This was the first cargo carried by any of the concrete ships built by the Shipping Board.

From Portland it proceeded empty to Baltimore, Md., loaded 2017 tons of coal, taking the same to Boston, Mass.
From Boston it proceeded empty to Norfolk, loaded 2028 tons of coal for Searsport, Maine.
Left Searsport empty for Norfolk, loaded 2025 tons of coal for Searsport, Maine. Delivered cargo at Searsport Feb. 6, 1920.
Left Searsport for Norfolk empty Feb. 7, 1920.
On the night of February 7, 1920, during a blinding snow storm and a severe blizzard the ship ran aground on Cilly Ledge and is still aground.

This grounding was due to the storm and poor seamanship. The material of which the ship was composed was not in any way responsible for the accident. A description will be given later in this article of the grounding and present condition of the ship.

"*Atlantus*".—The second concrete ship to be placed in commission was the "Atlantus," a 3000 ton D. W. cargo carrying vessel, 250 ft. in length between perpendiculars and 43 ft. 6 in. beam, 1400 H.P. engines, coal burning, speed 10½ knots. It was built at Brunswick, Georgia, during the summer of 1918 and 1919. The ship was given a sea trial off the coast of Georgia, then proceeded to Wilmington, N. C., for further work on the hull. The final sea trial was given, and the ship passed by the U. S. Steamboat Inspectors and classified by the American Bureau of Shipping. It was then placed in service and made the following voyages:

Dec. 1, 1919, from Wilmington, N. C., empty to Norfolk to load coal.
Carried first cargo on Dec. 21, 1919, consisting of 1917 tons of coal from Norfolk to Boston.
From Boston to Norfolk empty.
From Norfolk to Providencefi R. I., with 1988 tons of coal.
From Providence, R. I., to Norfolk, Va., empty.
From Norfolk, Va., to Boston with 2222 tons of coal.
Boston to Norfolk empty.
Norfolk to Philadelphia 2012 tons of coal.

Philadelphia to Norfolk empty.
Norfolk to Boston 1931 tons of coal.
Boston to Norfolk empty.
Norfolk to Boston 2030 tons of coal.
Boston to Norfolk empty.
Norfolk to Boston 2006 tons of coal.
Boston to Jacksonville, Florida, empty.
Jacksonville, Fla., to Boston, Mass., 2045 tons of lumber.
Boston, Mass., to Norfolk, Va., empty.

Owing to the excess number of Shipping Board ships in operation and to the decline in shipping business, the ship was tied up at Norfolk to await better conditions. The ship is at the present time tied up at Norfolk in first class condition and ready to operate.

"Cape Fear."—The third ship to be placed in commission was the "Cape Fear," a 3500 ton D.W. cargo carrying vessel, length 268 ft. between perpendiculars, 46 ft. beam, 1400 hp., triple expansion reciprocating engines, coal burning, speed 10½ knots. This hull was built at Wilmington, N. C., the machinery was installed at Jacksonville, Fla. The ship was given a sea trial, passed the U. S. Steamboat Inspection and was classified by the American Bureau of Shipping. The first voyage was made in December, 1919, from Jacksonville to New York with 2508 tons of railroad ties.

Then proceeded from New York to Norfolk empty.
From Norfolk to Panama Canal with 1807 tons of coal.
From Panama the ship proceeded to Chili and loaded a cargo of nitrate at Iquiquie.
From Iquiquie the ship proceeded to Martinique and discharged nitrate.
A cargo of mahogany logs was taken on at Cristobal and carried to New York.
From New York to Providence empty.

Left Providence for Norfolk empty and on the night of Oct. 29, 1920, she was rammed amidship by a heavily loaded steamer and was lost in Narragansett Bay. A description of the collision follows later in this article.

"Sapona."—The fourth ship to be placed in commission was the "Sapona," sister ship of the "Cape Fear," a 3500 D.W. cargo carrying vessel, length 268 ft. between perpendiculars, 46 ft. beam, triple expansion reciprocating engines, 1400 hp. engines, coal burning, speed 10½ knots. This hull was built at Wilmington, N. C. The machinery was installed at Jacksonville, Fla. The ship was given a sea trial, passed the U. S. Steamboat Inspection and was classified by the American Bureau of Shipping. The first voyage was made from Jacksonville, Fla., to Tampa, Fla., Jan. 20, 1920, empty.

From Tampa to New York with 2049 tons of phosphate.
From New York to Norfolk empty.
From Norfolk to Boston with 2074 tons of coal.
From Boston to Philadelphia with cargo of paper.
From Philadelphia to Providence with cargo of 2500 tons of coal.
From Providence to Baltimore, Md., empty.
From Baltimore, Md., to Boston, Mass., with 2900 tons of coal.
From Boston to Jacksonville, Fla., empty.
From Jacksonville, Fla., to New York with a cargo of lumber.

From New York to Norfolk Sept. 2, 1920, where ship was tied up owing to the excess number of Shipping Board ships in operation, and is awaiting an improvement in shipping conditions. The ship is in first class condition and ready for operation.

OIL TANKERS.

"*Selma.*"—The fifth ship to be placed in commission was the "Selma," a 7500 ton D.W. Oil Tanker, 2800 hp. triple expansion reciprocating engines, oil burning, speed 10½ knots, length 434 ft. 3 in. over all, beam 54 ft. It was built at Mobile, Alabama, during 1919 and 1920 and passed inspection by the U. S. Steamboat Inspectors. The first voyage was made from Mobile, Alabama, to Tampico, Mex., where ship loaded approximately 42,000 barrels of crude oil to be discharged at Philadelphia. The ship met with a grounding accident on the breakwater at the entrance of Tampico harbor and was temporarily repaired and taken to Galveston, Tex., to await permanent repairs. The grounding was due to faulty seamanship, the captain being intoxicated, and to the ship being operated without a pilot. The material of which the ship was built was in no way responsible for the accident. A description of the grounding follows in this article. The ship is now at Galveston awaiting appropriation of funds for repairs.

"*Latham.*"—The sixth ship to be placed in commission was the "Latham," sister ship of the "Selma," a 7500-ton D.W. Oil Tanker, 2800 H.P. reciprocating engines, oil burning, speed 10½ knots, length 434 ft. 3 in. over all, beam 54 ft. It was built at Mobile, Alabama, during 1919 and 1920, passed inspection by the U. S. Steamship Inspectors.

The first voyage was from Mobile, Ala., to Tampico, Mex., took on a cargo of approximately 42,000 barrels of crude oil.

Proceeded from Tampico to Philadelphia, discharged cargo and proceeded to Tampico, Mex., with a deck load of steel pipe for oil wells.

Upon arrival at Tampico the ship grounded on breakwater at entrance of Tampico harbor. The grounding was due to poor seamanship on the part of the master, who attempted to take the ship into this dangerous harbor without a pilot. The cargo was discharged and the ship proceeded to Galveston, Tex., empty where repairs were made to the bottom damaged by grounding. From Galveston, Tex., the ship proceeded to Mobile, Ala., where she is tied up at the present time.

The ship is in first class condition ready to operate. It is probably now the most complete of any of the concrete ships built, owing to the fact that several improvements and additions were made while the ship was undergoing repairs at Galveston. A description of the repairs follows later in this article.

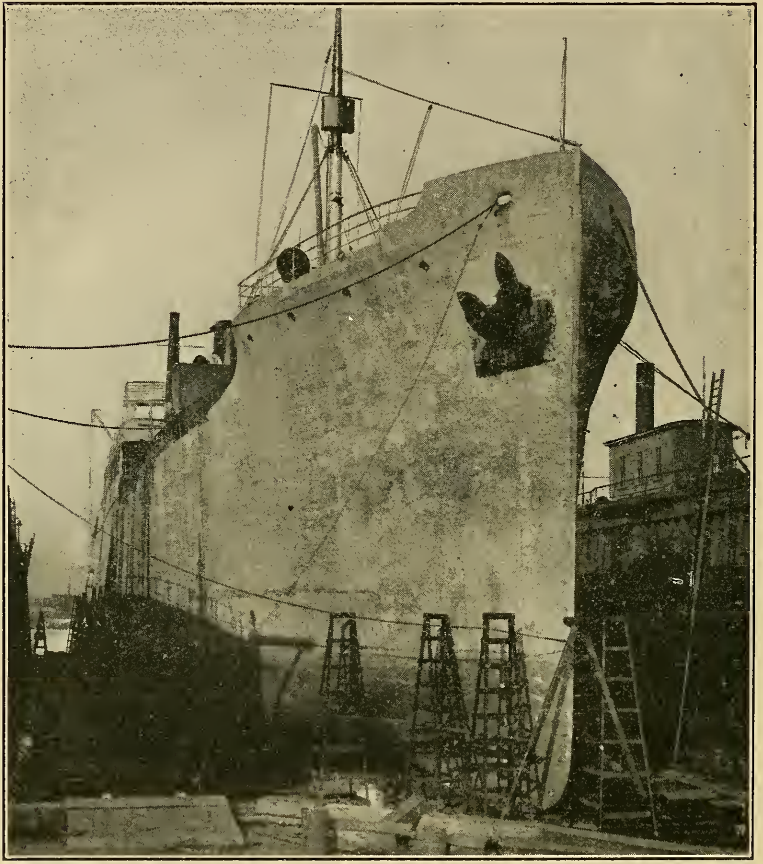

FIG. 9.—STEAMSHIP "SELMA," 7500-TON DEAD-WEIGHT CONCRETE TANKER IN DRY DOCK.

Neither the tanker "Selma" or "Latham" were given a sea trial.

They were probably the first ships in the history of shipping to be sent to sea without sea trials. The "Latham" did not even have a dock trial, the engine of the ship having been turned over for a period of less than four hours before the ship made her maiden trip from Mobile to Tampico. The rushing of the ships to sea without the customary preliminary trials was a grave mistake. This was done at the request of the American Fuel Oil and Transport Co., who had purchased the ships before they were fully completed. This company was anxious to get the ships in operation in order to carry out certain contracts that they had for the delivery of oil. This uncalled for rush in sending the ships to sea was unfair to the ships as they were experimental in a way, being the first tankers of this type ever placed in operation and should have been given the advantage of all preliminary trials given to other ships in order to obtain a fair comparison of their efficiency.

That the "Latham" made the trip from Mobile to Tampico and from Tampico to Philadelphia with a cargo of oil when the engines had only been previously in operation for less than four hours is considered remarkable and that more repairs were not necessary was only a matter of providence. This sending of the ships to sea in this unfinished condition was foolhardy and due to lack of shipping experience on the part of the American Fuel Oil and Transportation Co. Considering the adverse conditions under which the ships were operated a very creditable showing was made. Much better results would have been obtained had these concrete tankers been placed in the hands of older oil companies who had had experience instead of being placed with new companies who lacked experience in operating ships of any kind. Later on both the "Selma" and "Latham" met with grounding accidents at Tampico due to the mishandling by the officers placed aboard them by the American Fuel and Transportation Co. That neither of these accidents were due to the fact that the ships were constructed of concrete is shown by the statement furnished by the surveyor to Lloyd's Agent at the port of Tampico. The following is quoted from a letter written by him upon the subject:

"TAMPICO, MEXICO, 27 July, 1920.

WALTER R. HARPER, *Representative*,
U. S. Shipping Board E. F. C.,
Division of Construction,
CITY.

In reply to your inquiry of even date I would say most emphatically that the fact that the Strs. "Selma" and "Latham" are of re-enforced construction had nothing to do with the grounding of said vessels on the jetties at the mouth of the Panuco River.

Under similar circumstances and handling these vessels if constructed of either wood or steel would have met with the same mishaps.

With an experience of about thirty-two years in connection with the construction, operation and repairs of sea going craft of all descriptions, I feel warranted in saying that the "Selma" and "Latham" impress me as being well designed and constructed, strong in every requisite and properly found in every way. From information given me I would say that it is

> quite likely that the vessels would have been less liable to unmerited criticism had they been given more extended preliminary work-out before being hurried into regular service."
>
> (Sgd) SURVEYOR TO LLOYD'S AGENT.

"Cyamaca."—The sixth ship to be placed in commission was the "Cyamaca," sister ship of the "Latham," a 7500-ton D.W. Oil Tanker, 2800 hp. reciprocating engines, oil burning, speed 10½ knots, length 434 ft. 3 in. over all, beam 54 ft. It was built at San Diego, Cal., during 1919 and 1920, was given a sea trial and passed the inspection of the U. S. Steamboat Inspectors and was classified by the American Bureau of Shipping.

The first voyage was from San Diego to San Piedro, Cal., where ship was drydocked.
From San Piedro to San Diego empty.
On Oct. 1, 1920, the ship proceeded from San Diego, Cal., to Tampico, Mex., by way of Panama Canal. At Tampico took on a cargo of fuel oil approximately 40,000 barrels.
From Tampico to Galveston and discharged oil.
From Galveston to to Tampico empty.
From Tampico to Baton Rouge with cargo of oil.
From Baton Rouge to Tampico empty.
From Tampico to Galveston with cargo of oil.
From Galveston to Tampico empty.
From Tampico to Baton Rouge with cargo of oil.
From Galveston to Tampico empty.
From Tampico to Baton Rouge with cargo of oil.
From Baton Rouge to Tampico empty.
From Tampico to New Orleans with cargo of oil.
From New Orleans to Tampico empty.
From Tampico to New Orleans with cargo of oil.
From New Orleans to Tampico empty.
From Tampico to New Orleans with cargo of oil.

From New Orleans to Mobile empty, tied up March 15, 1921, owing to depression in shipping and excess number of Shipping Board ships in operation. The ship is in first class condition and ready to operate.

"San Pasquale."—The eighth ship to be placed in commission was the SS "San Pasquale," a 7500-ton D.W. tanker, length 434 ft. 3 in. over all, beam 54 ft. It was built at San Diego, Cal., during 1919 and 1920. Made a trip to San Piedro and was drydocked, had a sea trial, passed by the U. S. Steamboat Inspectors and classified by the American Bureau of Shipping. The ship proceeded from San Piedro to Tampico Oct. 22, 1920, by way of the Panama Canal. From Panama Canal empty to Tampico, Mex. It carried its first cargo from Tampico to Galveston, Tex., Nov. 19, 1920.

From Galveston to Tampico empty.
From Tampico to Baton Rouge with cargo of oil.
From Baton Rouge to Tampico empty.
From Tampico to New Orleans with cargo of oil.
From new Orleans to Tampico empty.
From Tampico to New Orleans with cargo of oil.
From new Orleans to Tampico empty.
From Tampico to New Orleans with cargo of oil, tied up at New Orleans. In first class condition and ready to operate.

"*Dinsmore.*"—The ninth ship to be completed was the "Dinsmore," a 7500 ton D.W. tanker, length 434 ft. 3 in., beam 54 ft. It was built at Jacksonville, Fla., given a sea trial, passed by the U. S. Steamboat Inspector and classified by the American Bureau of Shipping. It proceeded from Jacksonville to Mobile, Ala., where ship was drydocked. After docking ship was tied up at Mobile owing to the excess number of Shipping Board ships in operation due to the depression in shipping. The ship is now at Mobile in first class condition and ready to operate.

"*Moffitt.*"—The tenth ship to be completed was the "Moffitt," a 7500-ton D.W. tanker, length 434 ft. 3 in. over all, beam 54 ft. It was built at Jacksonville, Fla., during 1919 and 1920, given a sea trial, passed by the U. S. Steamboat Inspectors and classified by the American Bureau of Shipping. It proceeded from Jacksonville to Mobile where the ship was drydocked. After docking ship was tied up at Mobile owing to the excess number of Shipping Board ships in operation and due to the depression in shipping. The ship is now at Mobile in first class condition and ready to operate.

"*Palo Alto.*"—The eleventh ship completed was the "Palo Alto," a 7500 ton D.W. tanker, length 434 ft. 3 in., beam 54 ft. It was built at Oakland, Cal., and drydocked. It was given a sea trial, passed by the U. S. Steamboat Inspectors and classified by the American Bureau of Shipping. This ship was not completed in time to be put in operation until after the depression in shipping business came, consequently was tied up and it is now at San Francisco awaiting improvement in shipping conditions. At the present time the ship is in first class condition and ready to operate.

"*Peralta.*"—The twelfth ship to be completed was the SS "Peralta," a 7500 ton D.W. tanker, length 434 ft. 3 in., beam 54 feet. It was built at Oakland, Cal., and drydocked. It was given a sea trial, passed by the U. S. Steamboat Inspectors, and classified by the American Bureau of Shipping. This ship was not completed in time to be put in operation until after the depression in shipping business came, consequently was tied up and it is now at San Francisco awaiting improvement in shipping conditions. At the present time the ship is in first class condition and ready to operate.

The concrete ships have proven very staunch and seaworthy in the gales and heavy seas which have been encountered by the ships that have been in operation. Officers and crews who were at first skeptical and timid about sailing on them have after one trip been very enthusiastic over their behavior during heavy weather. The most noticeable feature about them is the lack of vibration of the hull and decks from the machinery. There is scarcely any vibration at all, which is very noticeable to a person accustomed to being aboard steel ships which are vibrating constantly. There seems to be no reason why this form of ship construction can not be commercialized and used successfully for light bulky cargo, such as cotton and general merchandise. With a light bulky cargo the earning capacity of a concrete ship is greater than that of a steel ship of the same deadweight tonnage, but with a heavy cargo the steel ship has the advantage. The weight of a concrete vessel makes it necessary to construct large hold spaces to obtain the required buoyancy. If advantage is taken of this increased space by selecting light bulky cargo this entire space can be filled and the ship still kept above the Plimsol mark while only a portion of the same cargo could be placed in a steel ship of the same tonnage owing to lack of space. On the other hand, with a heavy

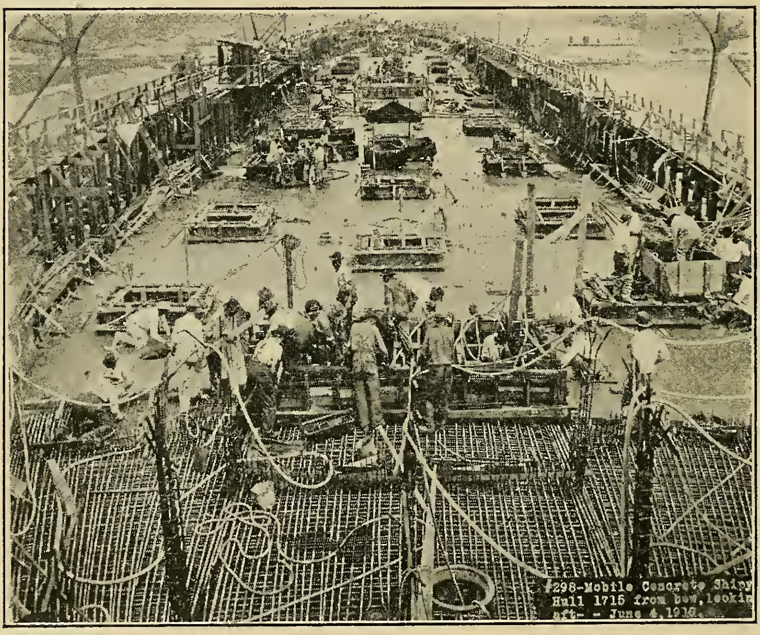

FIG. 10.—DECK OF CONCRETE TANKER PARTLY POURED.

cargo loaded on the concrete ship the Plimsol mark would be submerged before the entire hold space had been filled.

The advantage claimed for concrete ships over steel or wood vessels is that the original cost of building is much lower and they can be built in less time; that repairs can be made in much less time and at lower cost than on other vessels has been proven on the extensive repairs recently made on the concrete ship "Latham." They have no tendency to depreciate by rusting nor are they endangered by the corrosive action of salt water. They are not destroyed by toredo or other forms of sea worms which tend to destroy wood ships. The expense of drydocking at frequent intervals to scrape and chip the bottom is saved. Frequent chipping and scraping is absolutely necessary on a steel vessel. The entire upkeep of a concrete ship is far below that of other types.

Accidents.

"Polias."—There have been four serious accidents to concrete ships during operation, all of which have been found after investigation to have been due to faulty seamanship and were in no way due to the fact that the ships were constructed of concrete. The first ship to meet with an accident was the "Polias," a 3500-ton D.W. cargo vessel. The ship was proceeding empty from Searsport, Maine, to Norfolk, Va., and on the night of Feb. 7, 1920, encountered a severe blizzard and blinding snow storm off the coast of Maine. During the storm the master of the ship allowed her to get off the course and ran aground on Chilly Ledge, which is located just off Rockland, Maine. After the storm subsided the ship was left on the ledge in about 10 ft. of water and resting in a horizontal position. The bottom was damaged where it came in contact with the ledge, the main structure of the ship was practically uninjured and withstood the severe treatment. All damage done was local and at points on the ship that came in contact with rocks during the grounding. The ship is still in the same position on the ledge and in practically the same condition that it was immediately after the grounding, except that some damage has been done in cutting the decks in salvaging machinery. That the hull of the "Polias" is still intact and no indications of breaking up are shown demonstrates the great strength of the structure considering that it has withstood the ice and pounding of the storms of three severe winters off the New England coast.

On Feb. 7th, 1920, during the same storm that the "Polias" encountered, the "Princess Ann," a large steel vessel, went ashore off Rockaway Beach, L. I., and in a few days went to pieces due to sheering of the rivets of her plates. While the accident to the "Polias" was very unfortunate it at the same time has served to demonstrate the strength of concrete ships and their durability compared with steel ships under similar conditions.

"Selma."—The second concrete ship to meet with a serious accident was the "Selma," a 7500-ton D.W. oil tanker. The "Selma" while loaded

with a full cargo of crude oil in bulk was run on the rocks of the jetty of the breakwater at the entrance to Tampico Harbor, Mex. The accident occurred in daylight during fair weather and was due to poor seamanship, the master having been intoxicated attempted to enter this dangerous harbor without a pilot. After investigation of the grounding by the U. S. Steamboat Inspectors the master of the ship was deprived of his papers. The ship grounded on the jetty of the breakwater causing damage in the forward compartments, fore peak and dry cargo space. The master not being able to back the ship off from the rocks attempted to run over the obstruction and worked the ship ahead until it was aground just forward of the midship section with deep water under the forward and aft portions of the ship. It was impossible to move the ship from this position, where it remained for a period of twelve days, being constantly pounded by rough seas.

The ship showed remarkable strength and endurance while on the rocks. During this time it was loaded with a full cargo in the forward and aft oil tanks, which caused a cantilever action in the structure of the ship owing to the fact that it was aground amidships and in deep water under the fore and aft portions. This condition would naturally tend to hog the ship and break her back; that there was not the slightest tendency to hog is remarkable considering this severe treatment. Any hogging that developed would have shown immediately by cracks developing in the concrete slab of the deck.

After the ship was raised and removed from this position a thorough examination was made of the structure by the writer and no cracks of any kind were discovered, all damage having been done to portions of the bottom which came directly in contact with the rocks. The ship was raised from this position by installing air compressors aboard and pumping compressed air into the pipes for the steam smothering lines. These lines are connected from mains with each tank and are for the purpose of forcing live steam into the tanks in case of fire and for cleaning the tanks. Gages were installed connecting with each tank and a pressure of about 5 lb. per sq. in. allowed. Any greater pressure might have damaged the decks. When the air was forced into the tanks (the decks of course being airtight) it at the same time forced water out at the bottom through the holes made by the ship coming in contact with the rocks. In this way the ship was raised until it had sufficient buoyancy to be removed from the rocks and taken into Tampico Harbor where temporary repairs were made consisting of additional fastenings to hold the deck to the deck beams in case the ship should encounter rough seas while on the trip to the drydock at Galveston. In this conditions, with the water being forced from the oil tanks by pumping compressed air in them, the ship was taken from Tampico, Mex., to Galveston, Tex., four days at sea, and arrived in a satisfactory condition where she was tied up and is now awaiting an appropriation for permanent repairs.

Every precaution was taken to provide for the constant operation of the air compressors during the trip from Tampico to Galveston as the stopping of the compressors or cutting off of the supply of compressed air would have meant the sinking and loss of the ship while at sea. Five compressors were installed on the deck, two connected and forcing air into the tanks with the damaged bottoms operated by steam from the ship's boilers, two more compressors connected up and held in reserve ready to be put in instant operation in case of an accident to the compressors being used, and still another compressor operated by a gasoline engine, to be used in case of any trouble with the ship's boilers, making it necessary to shut off the steam operating the other compressors.

"Latham."—The third concrete ship to meet with an accident was the 7500 ton tanker "Latham." The "Latham" was proceeding on a voyage from Philadelphia to Tampico, Mex., with a deck load of pipe for oil wells. When Tampico was reached the master attempted to enter the harbor without a pilot and grounded the ship on the jetty of the break water only a few hundred feet from the point where the "Selma," sister ship of the "Latham," met with a similar grounding accident. At the time of the accident the ship was making about 10 knots and due to the error in her course was driven on and over the rocks of the northerly jetty of the break water instead of between the jetties. This caused four parallel gouges from 4 to 6 ft. wide to pe made on the bottom of the ship for nearly the entire length. No direct punctures were made, but the concrete in the gouges was crushed and held in place by the reinforcing steel in much the same manner as a piece of broken wire glass would be held. Notwithstanding this extensive damage to the bottom, the ship was able to proceed from Tampico, Mex., to Galveston, under her own steam and without a convoy. There were only three of the damaged tanks that could not be pumped dry with the ship's pumps.

Upon reaching Galveston the ship was drydocked and the work of repairs started. The damaged concrete was cut out; the reinforcing rods straightened; the oil removed from the rods and edge of concrete; wood forms were then placed and a new bottom poured in the damaged portions. Great care had to be taken to get bond between the old and new concrete. As the bottom of the ship also formed the bottom of the oil tanks, all surfaces were covered with crude oil, which had to be removed, as any oil in the joint between the old old and new concrete would have destroyed the bond. All shattered concrete was removed and the edge of the damaged portion cut back into the solid concrete until a firm face was made. (This work is similar to that of the dentist in preparing a cavity for a new filling.) All traces of oil were removed from the edge of the joint and from the reinforcing steel by sand blast, or by a steam jet. The dust on the new edge of the concrete, caused by cutting and deposit by the sand blast, was removed by going over the surfoce with an air hose with air at a pressure of about 60 pounds.

This removal of the dust between the old and new concrete is of great importance. After the forms were placed on the bottom the entire surface of the concrete joint and the forms were thoroughly saturated with water, in order that there should be no absorption of the water from the concrete before setting. Just previous to pouring the concrete, the joint which had been saturated with water was sprinkled lightly with dry cement. The concrete was rammed and agitated, in order to settle it around the intricate mass of reinforcing steel rods. By these careful methods an absolute water-tight job was obtained and when this ship was lowered from the dry-dock into the water there was not a single leak, or even a sign of moisture throughout the entire work. After repairs the ship made her first voyage from Galveston, Tex., to Mobile, Ala., during which voyage no leaks developed at sea, and the success of these extensive repairs was fully demonstrated. Both the "Selma" and the "Latham" were being operated by the American Fuel Oil & Transportation Co. at the time of these accidents, which were due to mishandling. These ships were sold to the American Fuel Oil & Transportation Co. and later returned to the Shipping Board.

"*Cape Fear.*"—The fourth concrete ship to meet with an accident was the "Cape Fear," a 3500-ton D.W. cargo ship modelled along the lines of the 3500-ton Ferris Type wooden ship built by the Shipping Board. The "Cape Fear" while proceeding from Providence, R. I., to Norfolk empty, with water ballast, was rammed amidship by a large steel vessel loaded with pig iron and sank in Narragansett Bay. The accident was due to poor seamanship alone, there having been a misunderstanding of signals. The weather was fair and the night clear and no excuse can be offered for the accident. The master of the ramming vessel was censured and his papers taken away after an investigation by the U. S. Steamboat Inspectors. The "Cape Fear" was empty and with water ballast. She was struck amidship and sank in a short time.

An opportunity has not been given to examine the hull to investigate the damage. The ship was constructed with the engine and boiler room amidships shut off from the forward and aft cargo holds by a water-tight bulk head at each end of the engine and boiler rooms and with a fore peak and aft peak tank separated from the fore and aft cargo holds by water-tight bulk heads. During the collision it is probable that when the ship was struck amidship that both of the water-tight bulk heads separating the boiler room and the engine room from the cargo holds were damaged allowing the water to flood the cargo holds and the engine and boiler rooms, and as the peak tanks were partially filled with water for ballast there were no unflooded spaces to furnish buoyancy to float the ship. Under these conditions it is not surprising that the ship sank in a very short time. Under similar conditions a steel ship would have gone down in the same time.

While these accidents may have had a tendency to prejudice those who were not familiar with the details against the ships, they have as a matter of fact in each case demonstrated the great strength and durability of the ships. The hulls have withstood severe treatment far better than ships of other types under similar conditions.

CEMENT OR GRANOLITHIC FINISH ON CONCRETE FLOORS.

By Joseph C. Grady.*

The purpose of this paper is to stimulate the endeavor to obtain the best cement finished floor possible for the varying uses to which they may be subjected and at the same time give due consideration to the following factors which must be studied and applied in each individual case.

1. Importance of the wear resisting qualities of the finish, to the owner.
2. Economy in the initial installation against permanence, and vice versa.
3. Dusting—very objectionable in certain kinds of manufacturing.
4. Appearance—immediate and future.
5. Level—generally important where machinery is to be set up.
6. Cost, difficulty and interference of making repairs.
7. Good floors to promote the use of concrete construction.
8. The contractor's reputation is very often fixed by the quality of his floor finish.

After considerable investigation and study of the question of finish for concrete floors, the writer is firmly of the opinion that a finish placed sometime after structural slab is in place, and in most cases after building is enclosed, will in general produce the most satisfactory final results.

Occasionally a certain combination of conditions will prevail in the field which will make the use of a monolithic finish very desirable, and at the same time produce equally satisfactory results. Likewise, a reverse combination of conditions may exist under which monolithic finish should not be attempted, as the results would be unsatisfactory.

Cement finished floors may be divided into three distinct types as follows:

A. Monolithic.
B. Finish placed lated (on structural slab).
C. Finish and base placed later (on structural slab).

"A"—Monolithic Finish.

Where a building is carried up even in warm weather at a speed of six or eight days per story, the final results will be scratched and scarred floors that are very poor in appearance and will not stand up under heavy trucking. On the same operation, should the element of speed be disregarded to the extent of allowing each section of finish to be properly protected and workmen kept off same for three or four days, an excellent finish would be the result. This mode of procedure would rarely be possible as the economy in design and material which belongs to this type of floor would be more than balanced by the interruption in the sequence of the various operations and the disrupting of the organization.

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*Construction Superintendent, Boston, Mass.

Exceptions to the above would be where a two-story building covers a considerable area and concreting of the first self-supporting floor can be kept three or four days in advance of the erection of roof forms, or where a building is erected in two or more sections and forms are moved horizontally.

This type of finish where excellent floors are required for heavy trucking should not be attempted when winter weather protection is necessary, as the cost is excessive and the results very uncertain.

Important Details.

1. Erect standard rain protection daily regardless of weather conditions.

2. Correctly proportion aggregates in rough concrete. An excessive amount of sand and cement will retard the water from draining off. Use only sufficient water to obtain a workable mix.

3. Rough concrete should be carefully graded to allow for a uniform thickness of finish material—¾ in. is sufficient.

4. Rough concrete of the proper consistency for reinforced work should be allowed to settle before screeds are set and before finish is spread. If this is not done and the finish is of proper consistency it will sink into the concrete and force the rough concrete to the surface. Pools of water on surface of rough concrete should be removed before finish is spread.

5. Run panels should be used to wheel finish over rough concrete. As each panel is removed, the smooth imprint left by the panel should be roughened and the displaced concrete brought to grade.

"AA."—MONOLITHIC FINISH. (Sprinkled Dry.)

This type of finish consists of "pulling up" the rough concrete by use of straight edge and short screeds clamped on column bars. When the water comes to the surface of the concrete, remove same by use of darby and straight edge. Sprinkle this surface twice with one to two dry sand and cement. The suggested placing of this dryer in two operations is due to the fact that the low spots which will require additional dryer are more easily detected after the first coat has been applied. The total of these dry applications should average about ¼ in. in thickness. When dry coats have become wet by absorption and surface will sustain knee boards, float and trowel in regular way.

This type of finish is recommended for floors which may be subjected to light wear or used for dead storage purposes.

"B"—FINISH PLACED LATER.

Except as noted under "A"—"Monolithic Finish," this type is recommended. The extra cost in design and materials is less than winter weather charges would be if the "A" type floors were laid during cold weather, and the final results are positive, while the "A" type results are uncertain.

Important Details.

1. Great care should be taken in levelling and grading the rough concrete slab, allowing a uniform thickness of ⅞ in. for finish to be placed later.

2. After rough concrets has been in place for several hours or has hardened sufficiently to sustain the weight of a man, use a steel wire push broom to sweep all fine material and laitance from the surface of the slab exposing the coarse aggregate. Do not use a rake.

3. Where floor finish is to be placed during cold weather, arrangements should be made to have the permanent heating system (if any) in operation before the floors are laid. Where salamanders must be used, sheet iron covers and flue pipes should be provided to carry coke gas out of building.

4. Prepare the rough slab to provide a bond for finish cost as follows: First. Remove all spots of mortar which have dripped from forms above, etc. Second. Thoroughly dry sweep well in advance of area to be finished. Third. In the afternoon of the day before finish is placed, thoroughly wash with clean water the area to be finished on the next day. Washing should be done with a hose with sufficient pressure to force out and carry away all sand and dirt or other foreign matter which may be lodged in pores or indentations of rough slab. The quantity of water necessary to properly wash the rough slab will often cause considerable trouble in the stories below where the washing is in progress. It is, therefore, advisable where either steam or air of sufficient pressure is available, to use same to blow rough slab clean. Acid need not be used except on spots of grease, paint, etc. After using acid, wash area thoroughly to remove all traces of resultant or spent acid. Sweep all excess water off the area to be finished. Do not allow excess water to stand on rough slab over night.

Before finish is placed, sprinkle the rough slab with water, but do not create a surplus of water. Next apply a 1 : 1 cement grout and sweep thoroughly into the rough surface. Apply finish before this grout dries.

"C"—FINISHED SEPARATE SLAB.

This type of finish generally consists of a 2-in. base of cinder or stone concrete with a 1-in. monolithic finish. This is the more expensive type of finish on account of the dead-load, loss of story height, and extra materials, but has advantages where it is desired to bury small conduits, pipes, etc., between the rough slab and the finished slab. (Steam and hot water pipes excluded.) Joints should be cut as described below and finish mortar allowed to turn down full thickness of slab and about 2 in. wide on all sides of each individual slab. This is to prevent curling. Screeds for this work should be set on dots the day before base concrete and finish are placed. These screeds are quite an advantage as a guide in placing the base concrete.

For best results, this floor should be laid in alternate sections, four slabs or sections to the ordinary bay.

Proportions for Finishing Coat.

One part cement, ½ part sand, 1½ part grit or clean hard rock (size ⅜ in. down, with sand or dust removed).

In the finish or topping, use the minimum amount of water that will allow the finish to pull with a saw action on the straight edge without tearing apart, leaving a fairly smooth dense surface.

Finishing.—Floor should be floated and troweled twice. The first operation should not take place until the finish has set enough to retain the knee boards on the surface. Floating and troweling of wet finish settles the coarse aggregate and brings fine material to the surface.

Floor Joints.—Mason's jointer should not be used on any floor which may be subjected to trucking. The joint should consist of a knife cut only.

"A" type floors should have knife cut joints at fill lines only.

"B" type floors should have knife cut joints at fill lines and in finish directly over construction joints in slab.

"C" type floor should have knife cut joints at fill lines, that is, at column centers and at center of bay in each direction.

After Protection.—Cement finished floors should be kept saturated with water for two weeks after being laid. Water must not be applied until floor is hard enough to prevent blistering and peeling. This is wholly a matter of judgment and is dependent on atmospheric conditions.

A 1-in. layer of sawdust spread on the floor and kept thoroughly wet with water will assist in holding moisture, and properly protect and "cure" the floor.

A strip of burlap 3 in. or 4 in. wide laid at edge of preceding day's work and overlapping sawdust will help to prevent sawdust from being blown or dragged on to new work.

Hardeners.—Surface or after-applied hardeners will add somewhat to wear-resisting qualities of a floor which is subjected to grinding action from heavy trucking, that is, provided the floor contains the proper hard aggregate and is what might be called a good floor to start with. Under occasional very light trucking or foot traffic—many of these after-hardeners are very beneficial, and in either case are more or less effective in preventing dusting.

Integral hardeners and accelerators are recommended for use in Type "B" and "C" finish, provided no mesh or reinforcement is to be placed in same. This type of hardener is also recommended for monolithic finish when used in connection with plain concrete, but should not be used in monolithic finish or reinforced concrete until more favorable information as their action on metal is available.

Manufacturers' recommendations as to quantity to be used should be followed, except in very warm weather, when the amount may be reduced. This reduction in amount is wholly a matter of judgment and is dependent on temperature, etc.

MISCELLANEOUS ITEMS.

Where artificial heat is used in connection with finish floor work, a temperature of about 60 deg. should be maintained. Too much heat will cause floor to dust badly when dried out.

Placing of all floor finish should be arranged so that it will be unnecessary to wheel over newly laid work. Where it is necessary to wheel over "Green" finish, great care should be taken to prevent damage to floor already finished. This can be cared for as follows:

Place planks or timber on building paper parallel to the runway and with a bearing directly over either beams or girders. These supports should be placed a sufficient distance apart to allow for a spring action in the 2-in- plank cress supports directly under runways.

TEST TO DETERMINE WEAR RESISTING QUALITIES.

During the early part of the present month the Turner Construction Co. carried out an extremely accelerated and concentrated test on a section of permanent floor finish (not a sample). The finish on which this test was conducted was of the "B" or "placed later" type which had been in place five and a half months, and was installed in accordance with the foregoing recommendations for this type of floor.

The testing apparatus was arranged in a manner to concentrate in a given path the maximum grinding action that could possibly be developed with many years of extremely heavy factory or warehouse trucking.

Testing Machine.—Four wooden boxes each mounted on two 10 in. diameter by 3 in. face iron wheels were rotated about a vertical shaft which was driven by bevelled gears and 10 hp. motor. These boxes were loaded with sand and held on a 6 x 6 in. radius measured to center line of trucks, by wooden arms, constructed with a vertical knuckle in each arm to guard absolutely against any lifting action.

The total weight on each pair of wheels was 635 lbs. The axles were fixed, not pivoted, thereby causing a severe grinding action on the floor while turning on the short radius. The speed of these loaded trucks was regulated so that a pair of wheels passed over any given point on the circumference of the circle 1740 times per hour or a total travel of 269 miles during the twenty hours that the machine was in operation. This travel was measured at the center line of the trucks. The maximum wear at any point was about 1/8 in. and the floor was left in practically an uninjured condition.

In order to determine the value of the knife cut joint which was referred to under "Joints" this testing machine was centered on and operated over a knife cut joint which was also a fill line between finish placed on different days. The result obtained after the twenty hours of continuous grinding over this joint was very gratifying. The joint remained in perfect condition without the slightest sign of crumbling on either side.

On the work where this test was conducted Quickstone Liquid Integral Hardener was incorporated in the floor finish mixture.

KALMAN CEMENT FLOOR FINISH.

Cement floors are installed by the Kalman Floor Co. using the basic principals covered by the Bruner patents which that company acquired from Preston M. Bruner, of St. Louis, Mo. The method is as follows:

Several hours after the structural concrete slab or base is in place, the exact time being dependent upon the atmospheric conditions, the surface of the concrete is prepared to provide a bond for the finished coat which is applied later.

After the slab has thrown off its excess water and at a time when the concrete will crumble under a steel brush, rather than smear, the surface is well brushed with wire brooms, all fine material and laitance are swept from the surface exposing the coarse aggregate.

On the day before finish is placed, all the area to be covered is thoroughly washed with water and the slab kept wet until the finish is to be applied. Immediately before finish is placed, dry cement is sprinkled on the wet surface of the rough slab and a broom is used to sweep this cement into a grout which fills all the pores and crevices in the rough slab.

Screeds are set to established grade and the concrete which is a mixture of 1 part cement, 1 part sand and 1½ parts pea gravel or granite chips is rodded off with a straight edge in the usual manner. The finish coat is mixed to a consistency which will permit of economical rodding. Mixing is done by one bag mixer set upon the floor adjacent to the work. This mixer is hoisted from floor to floor as the work progresses.

Immediately following rodding, the surface is covered with a loosely woven burlap in 3 ft. widths, over which a dry mixture of equal parts sand and cement is spread to a depth of from ¾ to 1 in. in thickness. This dryer acts as a blotter, absorbing the excess water in from 15 to 20 minutes.

In about 20 minutes after the dryer is spread, the burlap strips with the now moist material are lifted off and this moist material returned to the concrete mixer where the proper amount of coarse aggregate is added, for use in the following batches.

Immediately after the burlap is removed the surface is floated with a heavy electrically driven impactor which brings sufficient moisture to the surface to permit of troweling by masons. This mechanical float which weighs 110 lbs. is driven by a ¾ hp. motor and is easily operated by one man.

Ordinarily, two trowelings in addition to the impacting, which is done by the motor-driven float, complete the work. The time required under normal conditions is about 40 minutes from the time the finish is first placed until it has been floated with a mechanical float and received one hand troweling. The final troweling or polishing usually takes place within three hours.

The dry material used on the last section of burlap placed in each day's work must be wasted unless some other masonry work in which same can be used is under way at the same time.

A layer of sawdust approximately 1 in. thick, kept damp for about ten days by frequent wetting is used to protect and "cure" the completed floor. This treatment is generally applied on the work that was completed on the previous day.

A liquid floor hardener is applied when floor is dried out.

Following are a few points covered by the Kalman method which are theoretically correct and are very important:

a. The absolute control of the water ratio accomplished by absorption.

b. All necessary impacting, floating, etc., except the final troweling, can be done before cement takes initial set.

c. By the use of the mechanical float or impactor, a very dense floor surface is secured, and a greater percentage of coarse aggregate can be used than would be practicable by hand method .

A SPECIAL TYPE OF FLOOR FINISH.

By W. M. Bailey.*

Before laying the type of floor finish described in this paper, a number of tests were made on various kinds of concrete surfaces to determine the one having the greatest resistance to wear and best adapted to the severe wearing conditions occurring in industrial plants. The problem was considered with reference to (a) materials incorporated in the finish and (b) method of laying the finish.

In order to obtain as nearly as possible conditions encountered in commercial practice, a testing machine was made consisting of four boxes mounted on small steel wheels, attached to wood arms held in a central vertical shaft. These four boxes were driven by a motor so that they revolved on a circumference with a radius of approximately 7 ft. The boxes were weighted with sand and tests to destruction were made on a number of different kinds of floor finishes. Under these severe tests the ordinary granolithic finish showed signs of wear within five minutes after the tests were started, while the sample of finish finally selected (which has come to be known as the Bailey Finish) stood up for over thirty hours without appreciable signs of wear.

Attention is called to the fact that the wheels on the sand boxes were not pivoted in order to produce rotary grinding similar to the effect of heavily loaded tractors and trailers in turning. This action had a far more destructive effect than the general wear from straight run. The test also showed that quartz particles in the sand on the ordinary granolithic finish oftentimes were broken out causing a pitted surface. This is probably due to the quartz being fractured along the planes of crystallization.

The material for floor finish to be described consists of 1 part cement and 2 parts trap rock, graded in size to produce minimum interstitial spaces in order to approach as nearly as possible a solid trap rock surface. The general appearance of this type of finish resembles terrazo except that the area of voids is greatly reduced. Special attention was given to the method of laying the finish in order to insure against dusty surface which is often encountered in granolithic surfaces. In order to compact the aggregate and reduce the interstitial spaces to a minimum, the finish was subjected to successive rollings as described below. The rolling also forces out excess water, laitance and other impurities. Grinding was done solely to remove the hardened film on the surface.

The finish aggregate consisted of two sizes of trap rock passing through ¾ in. and ⅜ in. rings and retained on ⅜ in. and ⅛ in. rings, respectively.

The mode or procedure of laying the finish is as follows:

a.—Surface of floor slab shall be carefully prepared to insure perfect bond.

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*Boston, Mass.

b.—Mix dumped on floor and screeded. The consistency of the mix should be such that it will stand in piles, when dumped from buggies.

c.—Screeds removed and additional aggregate sprinkled over the entire surface to decrease voids to a minimum. This additional stone is troweled into the body of the mix by means of steel trowels, used flat, and wood floats. After about one hour, depending on the set, the finish is ready for rolling.

d.—The initial rolling is done by a concrete roller with a width of about 2 ft. and weighing about 50 lbs. per lin. ft. Surface is rolled both lengthwise and crosswise confining one series of rollers to each direction.

e.—In the second rolling, rolls weighing 100 lbs. per lin. ft. are used. This is started when the finish is sufficiently set to avoid breaking in front of roller.

f.—The third and last step in the rolling process consists of using a roller weighing 350 lbs. per lin. ft. and 4 ft. 6 ins. in length. This rolling compacts the aggregate into position, reducing the void area to a minimum and forces out all excess cement. Any unevenness in the surface left by the rollers is smoothed out by steel trowels.

g.—After about fourteen hours depending upon the temperature of the air, the floor is flooded with water to a depth of 1 in., and the water is kept at this depth for about four days.

h.—Rotary motor-driven grinders are used to remove the hardened scum, laitance, etc., on the surface. Sand is used to keep the scum from sticking to the grinding blocks.

i.—Floor is washed with clean water after which it is ready for traffic.

This type of floor has the advantage of eliminating much overtime work. Fourteen thousand square feet of this finish have been laid in one operation. Although the unit cost is probably slightly in excess of granolithic, the greatly increased wearing qualities of the finish described in this article over the ordinary granolithic more than offset any excess unit costs.

It is felt that, in future, as much attention will be given to "permanency" in concrete floor finish as has been given to the importance of this term as applied to concrete construction in general.

SMALL CONCRETE HOUSES AT ROCHESTER, N. Y.

AN INFORMAL TALK BY KATE GLEASON.*

I always like to talk about these houses, but particularly to you people, because I think you will sympathize with me. The reason I particularly like to build houses to sell for about $4000 is that the people who are interested in that price are the finest people to do business with I know. The ones who want the $10,000 and $12,000 houses are so hard to suit that I haven't the time to try; and the people who want cheaper houses are usually rather slip-shod, do not appreciate them and do not try to improve them.

My selling price without the lot is about $3600, but that has to include a certain price for marketing, and it also costs something to collect when I take only $400 down and $40 a month, then the interest and taxes are taken out of the $40, and naturally that extends over a period of years and needs bookkeeping; so if you people were to build the houses for $3200 each for somebody who would pay you for them and sell them, you could make money on them, I mean probably a good deal more than I do because you have better organizations and you have been longer in the business.

The reason why I took to poured concrete is that I have a remarkably fine sand pit within a mile of the property, and then another thing, Mr. Ransome, of the Ransome and Smith Co., pioneered in our locality. He put up the American Piano Co. building about 18 years ago, and he educated the countrymen around there to concrete construction so that they all know how to do it and do it very well.

One reason, of course, that my price comes low is this sand bank; I do charge sand to the job at $1.50 a yard and make money at that. If I had to, I could deliver it at $1.00 a yard, and I do not suppose many of you can get good sand at that price. Then another thing, my wages average 50 cents an hour, counting the foreman, the skilled help, plumbers, electricians, and everything else along with the ordinary men.

I am going to give you a little idea of what, it seems to me, is economical equipment. Chimney and drain tile forms—that is a handy thing to have; in bad weather the men can be putting ahead those tiles. And then a saw table with an electric motor. The trouble about a gasoline motor is this, that in cold weather it is hard to start. About four portable sheds, that one can move from one place to another—they are good for concrete and for all kinds of rough things. One portable office and tool house. Two Ford chassis. You do not need any chauffeurs if you have a Ford chassis, anybody can run a Ford; and so, when a man wants something, he simply lays his hand on the Ford and goes after it and gets it and he is back before you know he wants it.

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*Rochester, N. Y.

Then you want with that about three one-ton trailers. I can take my cement from the railway switch to the storage shed, half a mile away, for a cent a bag. You see the 1-ton trailer is there, the men are loading it, then the Ford hitches on and takes it over to the storage shed; the men are there to unload it. The driver leaves the loaded trailer, hitches on to the empty one and takes it back, and in the meantime the men at the car are loading the other; it is a continuous procession; there are only four men and the driver needed to unload that whole car. We have unloaded a car of cement in three hours with four men.

Then we need another trailer equipped with a plumbing and electric outfit, cutting tools and vises and threading tools and things of that kind. You move that around from house to house. Then there is the concrete lifter. We made our own because our work is small and we find that we get through all our pouring in fifteen hours for a complete house; that includes the first and second story walls, the stairs, the stair-well, the chimney complete, the floor, the beams and the columns. And then you have to move your machine. If you have one of these heavy machines, it takes a day and a half of 5 men to move it. We can move the telescopic machine that we made on our job in two hours with 3 men and a team. I know it would not pay for anything but a light job, because we cannot lift more than a yard with it to advantage, in fact we get along a little faster with our 7hp. hoist if we haven't the bucket quite filled; it sometimes jerks when you have too big a load in it.

Then we want about two sets of steel stairforms. They are the only steel forms we use, and the stairs include the stair-well. We fill all forms in one job. We use steel plates for the floor form, because we use open reinforcement, pour through this open mesh on to the steel plates, which gives us a smooth finish and all we have to do is to give it a little stoning, to have our ceiling all done, with no plastering to be done to it. Three sets of wood wall-forms, and these we make ourselves. Three sets of beam and column also of wood. Three sets of chimney forms, also of wood. They are easy to move. I tried steel forms, and for our job where we require to move them so often, we found they were very clumsy, in fact we only tried to move them once.

Then two sets of ladder scaffolds. That is something I found over in France. I had tried the steel scaffolds and the trouble about them is that the men let them go smash when dismantling them and they all bend into knots and you never can get them up the way they were to start with. You use a pair of extension ladders and just the braces on the side and as many sets as you need to extend across the front of your building. They are very easy to move and they do not run much into money and they will last all the way through. The men can use them for the roof as well as for the stucco, in putting on the conductor pipe or anything that you need a scaffold for.

I would arrange the portable sheds and the shed over the saw table to form a hollow square. Put your lumber and other material in the hollow square, and you won't lose so much of it.

The equipment will run about $6,000 to take care of about 25 houses, which is a very economical number to build, as, on the average, it pays to get all your houses done and all your clean-up work done before you let any tenants move in. If you let a tenant move in, they have a talent that amounts to genius for thinking something needs to be done to their house while the men are around.

When it comes to the design, my special preference is for one general design. I received that preference when I saw the World's Fair, which was on the Greek order; then take that wonderful development in New York, Forest Hills, old Nuremburg style, and then there is Venice, in Italy, on the Renaissance style, and all those little English villages. It is also a more economical form of building, and it seems to me a more pleasing one than to run in the Queen Anne and Dutch and Colonial and mansard roofs, etc., all within a few squares of each other.

It is a very good practice, I find, to avoid grading wherever possible and follow the contour of the land; it helps on the cost and on the appearance. Mr. Whipple has advertised for me here in Cleveland that I tried to please the ladies with a mirror and a powder puff, but I try harder to please the men and put in a garage that is warm on these cold mornings, and provide the garage with doors and hangers that cost enough to put mirrors and powder puffs into forty houses; so you see I spend more money on the men than on the women.

I put in a small kitchen. I copied that from the Pullman diners; they serve three hundred people in a space of about 2 x 3 ft. and in order to show it could be done I equipped my kitchens with stoves, with special kitchen cabinets, with a very special mixing faucet that usually goes into very expensive houses, and a refrigerator and laundry tray. I explained to the women that if they want to go to the movies, all they have to do is to shove the dishes in the laundry tray, put down the cover and forget them, and may be if you soak them, in the morning they will be clean any how.

Another way that I economize that seems to me is money saved to good advantage, is making no cellars. For one thing, that saves plumbing, you see, you at once save about 8 ft. of 4-in. soil pipe any how, and you save on the sewerage; you do not have to go down so far to get to the sewer. It saves an extra stairway. It gives a great deal more bedroom space up above, if you put your garage out at the side and use it as you would a cellar.

Now about the insurance. That thing of putting the garage in the house only costs 15 cents a hundred extra for insurance, and it is not necessary, of course, in these houses to have any insurance unless one has a mortgage on it, then those mortgage owners certainly are very particular.

One of the things that it seems to me is economical is to let the length of your house be what you will, but keep to a 10-ft. span. If you have ever noticed the tables issued by the reinforcement people, they give a great deal less weight of reinforcement necessary for those small spans than they do when it goes up. I can buy reinforcement that will give 70 lb. per foot, which is more than most building codes require, on a 10 ft. span. Well, it really is not a 10 ft. span, it is 10 ft. from outside to outside, 9 ft. 2 in. between bearings. I can buy that reinforcement at the present time for 5 cents per square foot, cut to size, and if you are buying it for over that, you know it runs way up. Then another advantage that short spans give you is your roof, two by fours is all you need in your roof.

Then another economy, it seems to me, is in the ceiling heights, 7½ ft. from floor to floor, from the top of the floor to the top of the next floor; that is 7 ft. 2 in. in the clear. You get away with 11 treads on your stairs, instead of the customary 17; that takes less room and requires less money. It is a great advantage with the pipeless furnace, because it does not give a chance for the heat to stay up there, you get circulation, and the pipeless furnace is certainly the one, for with this we can heat our house 20 x 30 ft. with the garage, with a temperature sometimes 10 deg. below zero, with five tons of coal per year. I have been told that the fashion of 9 ft. ceilings for small houses dates from the time of gas chandeliers which had to come down pretty low in the room in order not to burn the ceilings, and that does not apply to the present electric light. Then too 7 ft. 2 in. gives you a better looking room. I you have a bedroom 10 ft. square, it really makes the bedroom look quite large. I have surprised a good many people by telling them that the 20 x 30 ft. houses, two stories, cost me so much less money than the 20 x 20 ft. three stories. I have not yet built a one-story house, but I feel that in the concrete one story is going to be the most economical house. A one story house can have lighter foundations and lighter walls and will take still less time to pour, there is less plumbing and no stairway, and I have been amazed to find how little extra the extra space in the roof costs, it does not cost anything more than the added material; the men get through with a 20 x 30 ft. roof in the same time as they do in the 20 x 20 ft. That is explained by the fact that the fitting is all on the corners; straight work does not take long.

One thing I find costs a great deal of money when one tries to vary the style of the roof is valleys; I do not put any valleys in. If I want to add to the appearance, it seems to me to be a more economical, a more pleasing way, to buy tile or colored slate or something of a permanent value that only goes on the high priced houses. It is a curious thing, you can put one little touch on a $4000 house that usually goes on a $10,000 house, and the people who live in it have the feeling that it is a $10,000 house.

There is something I adopt in my house that I am surprised is not more generally used, because you will notice house plans seldom show it, and that is one-line plumbing, have the bathroom and kitchen either side by side or one directly above the other. And a great difference can be made in the cost of the plumbing by putting the head of the tub, the closet and basin right in line instead of putting the tub on one side of the room and the other two things on the other. Every time you save $5.00 apiece on 25 houses, you save $125; that is one of the beauties of building them in quantity.

I have to tell you once in awhile of my mistakes, and one of them was in trying to get an artistic effect on these small houses by adapting Frank Lloyd Wright's plan of the panels under the windows. No doubt Mr. Wright had some reinforcement to make up for that, and all I did was to copy it from a picture without paying him any commission, so naturally I had to pay the price somehow, and the price I found was that the 3 in. under the window shrunk faster than the 4 in. in the adjoining wall and I would be apt to have a crack right there under the window; so you want to keep the wall equal thickness; at least I do, after this.

Another trouble I found was putting sills under the windows; it makes another joint for the wind to drive the rain through when it is strong enough to do any driving. Then it pays better to slant your windows at the top and the bottom, so that the rain will have to drive uphill; it is not so easy for it to do that as to go straight in.

One thing that has been successful with me is letting contracts for a good deal of the work, for instance, putting up the forms. In a contract job the men feel as though they are in business for themselves. I make the contract to take down the forms, move them to the new place and put them up, and it is understood that the place it is to be moved to is only the adjoining house. If, for any reason, we would have to move them 100 ft. or more, I would have to do the moving on our own time.

I used to erect one story at a time, first a wall and then a floor and then a wall and then a floor and then the third wall. There is a disadvantage in that you get an uneven texture on the outside that is apt to show, so this year we make our wall forms in vertical panels and erect the two floors at once, the two stories, rather, at once. We also put in our floors, but we only pour a story at a time. We leave a space after the first story in which we can pour the wall, then we pour the floors just as though they were erected separately, but the result is one mass when you are through without costing any more to do your pouring. We also have to promise our contractors who move the forms for us, that we will furnish them a foundation absolutely level. They do not have to do any adjusting on their forms; that is attended to, and we save a great deal of money by that and get a better looking place as well.

It also pays us to furnish a house as we think it ought to be furnished; we keep one sample house. I have in mind now the furnishing of 25 houses which I am building.

DISCUSSION.

Mr. Boyer.

E. D. Boyer.—How do you finish the interior walls of the house?

Miss Gleason.

Miss Kate Gleason.—At the present time I am using on the nailing strips, fiber board which I buy directly from the mill. It is the same class of board as that sold by the dealers, and is guaranteed to withstand 350 lb. pressure per 12 in. I have seen it under the testing machine. It forms an air space inside the poured wall, but of course it is not good to look at, so on top of that we put 8 oz. duck, on frames so we get two air spaces, the one between the fiber board and the concrete and the one between the fiber board and the canvas; then the canvas is our finish.

Mr. Boyer.

Mr. Boyer.—So that you have no condensation on the interior wall?

Miss Gleason.

Miss Gleason.—No, the only place we have to look out for trouble is around the window frames. The windows open out, and I do not have any trouble now that I have got the sills right. There was a chance for the water to blow in under here and up above here, and then there is a shoulder inside the window so that the water does not get in through the frame itself, and we tip it down here a bit and do not have any sills. The water did not enter but one house, but it might happen on others, so we did not take any chances on it happening at all. I am very much in favor of those French windows, because they give so much ventilation without taking up wall space; you get as much ventilation as you do with a double window, by the time you run one up against the other you have only got one opening anyhow, and with this you get the full space of the opening and all this space below.

A Member.

A Member.—How are the attic floors finished off? Is the attic floor concrete?

Miss Gleason.

Miss Gleason.—No, I make it of wood. I expect to make a change perhaps next year, but I am making all the roofs of wood with asbestos shingles.

A Member.

A Member.—What is the finish on the outside?

Miss Gleason.

Miss Gleason.—That is stucco. We have no better way to put it on than with twigs; I have tried using the machine, but it is too heavy for our men, and there isn't any way we can lighten it up, but we figure on getting a plant; I have thought that something on the order of those hand sprays that you use for the gardens, with a little more power, might do for it, but we have not had a chance to try it yet.

Mr. Boyer.

Mr. Boyer.—In other words, you just dash on one coat on the finished structure?

Miss Gleason.

Miss Gleason.—Yes.

A Member.

A Member.—That is cement and stucco, Portland cement and stucco?

Miss Gleason.

Miss Gleason.—Yes, that is what it is. Do you want me to tell you what we do in the way of reinforcement? We reinforce a rod around every opening no matter what size it is, simply to get away from cracks,

Miss Gleason. not with any idea that it strengthens it. We reinforce over the garage doors and in the columns and in the beams, and we put one rod around the roof line, to tie it together. I do not know that that is necessary, and sometime I may make an experiment and leave it out, because the roof itself holds it very well, but we do not put any reinforcement in the walls. A 4 in. upper wall without reinforcement is absolutely all right. You never have the sign of a crack or settlement in them.

A Member. A Member.—How rich do you mix your concrete?

Miss Gleason. Miss Gleason.—Six to one. We have no stone, we use simply the sand except for the floor; we use a richer mixture for the floor, but I have heard that you could use as much as ten to one, but we have not tried it, because with the small quantity we use we might as well use the six to one. The floor is reinforced; I use the welded wire at the present time, we can get that cut for the 24 ft. lengths that we want, and it comes very open mesh so that we can form the ceiling of the lower room at the same time we form the floor of the upper room, and we finish as we go along in a rough sort of way, about like you would a sidewalk finish for the upper floor, and then put a linoleum over it to cover it, because we find that the more we eliminate hand finish in plaster or floors or any other part of the house, the better, and depend simply on machinery; you get some good men who will do hand work well for you, but it costs a great deal and then once in awhile it won't come right.

A Member. A Member.—Will you explain your method of making your floors dustless?

Miss Gleason. Miss Gleason.—We put linoleum on them.

Mr. Whipple. Harvey Whipple.—How did you make your floors dustless before then, before you promoted that finisher to be a foreman?

Miss Gleason. Miss Gleason.—We rubbed in a wax made by the Vacuum Oil Company. It is ordinary oil wax, paraffin wax, and that has a certain amount of oil in it, and we ground that up in a meat chopper, and dusted that on and then took an electric machine to rub it in. We used to make the floor, when we put the finish on, about two to one of cement, with a good man to do it, made a very firm mixture, but the trouble is that when that particular man was off, the next man would put an everlasting amount of water in, it was easier for him and did not make as good a finish, so it was always dusting, as you say, and we would have to re-finish it and it would make trouble, so we gave it up, we do not try to finish any more.

Mr. Boyer. Mr. Boyer.—Is that included in the cost of the house?

Miss Gleason. Miss Gleason.—Oh yes; linoleum is not very expensive.

A Member. A Member.—How do you finish the linoleum?

Miss Gleason. Miss Gleason.—We do not try to finish that; I get the inlaid linoleum. Of course it is better naturally for the owner to varnish it or shellac it, but when they buy the houses, they will do that.

A Member. A Member.—How do you fasten the linoleum down?

Miss Gleason. Miss Gleason.—Just lay it down.

A Member. A Member.—You do not cement it down?

Miss Gleason. Miss Gleason.—We don't have to.

GUN-STONE HOUSE AT WATERTOWN, MASSACHUSETTS.

By H. Whittemore Brown.*

A concrete house has recently been constructed in Watertown, Mass., that is the result of considerable research on the problem of constructing a house which would combine as many of the following principles as possible:

1. Maximum protection from the elements.
2. Economical use of materials.
3. Indestructible by fire or elements.
4. Attractive interior surfaces.

The result of this research is the system of construction which has been termed "Gun-Stone," because of the combination of gunite and concrete.

The floor plans and perspective elevation of the Watertown house are shown in Fig. 1.

The walls are made up of an exterior covering of approximately 1½ in. of gunite supported by wire or expanded metal reinforcing and attached to a poured concrete frame. This frame is made up of sills, studs, and girts somewhat the same as in braced-frame wooden construction.

Between the concrete studs there are light wooden panels which have waterproof burlap or tarred paper next to the gunite, and wooden laths on the inside as a support for interior plaster.

At the studs the inside of the concrete is covered with tarred paper; metal lath is nailed to the wooden panels on either side, covering the space between the panels. The whole interior surface is plastered in the ordinary manner. Thus every portion of the interior plaster is insulated from the exterior concrete by tarred paper and by an air space. This air space is about 6 in. between studs and about a ¼ in. at the studs.

The floors are similar in principle to the exterior wall: burlap covered panels support a poured slab, while the space between panels forms the stem of a T-beam. The plaster is insulated from the concrete of the floor in the same manner as in the walls to eliminate any possibility of staining.

The interior partitions are of non-bearing wood stud and lath construction; all the interior load is carried on concrete girders and columns.

The roof is made of wood, with plates anchored to the concrete of the third floor. The covering is asbestos-backed asphalt shingles. A wooden roof was used on this house because the third floor was not required for rooms, and a hip roof was necessary only for appearance and for insulation. With the exception of the roof, every structural member of the house is made of concrete. Wood is only used where protected from fire and moisture by concrete or asbestos. Even if the roof should get afire, the contents of the house would be protected by the concrete third floor.

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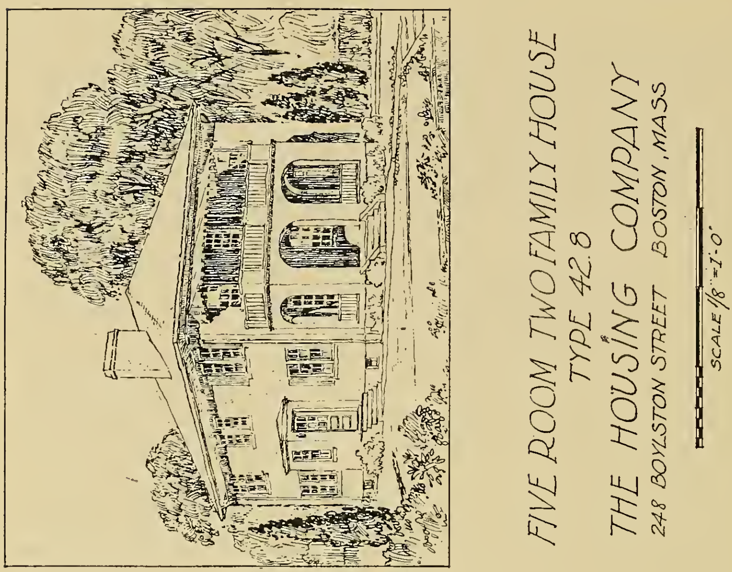
* Boston, Mass.



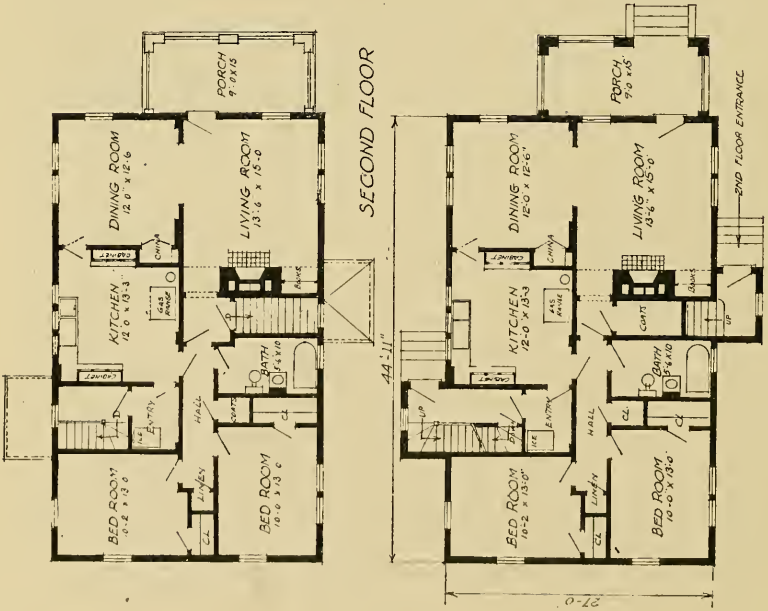


FIG. 1.—PLAN AND PERSPECTIVE OF HOUSE.

The house is finished with ordinary wooden trim nailed to the wooden panels. In the lower apartment wooden top floors are nailed to the concrete, excepting in the kitchen and bath, where linoleum is used. The upper apartment has linoleum floors throughout, glued directly to the concrete.

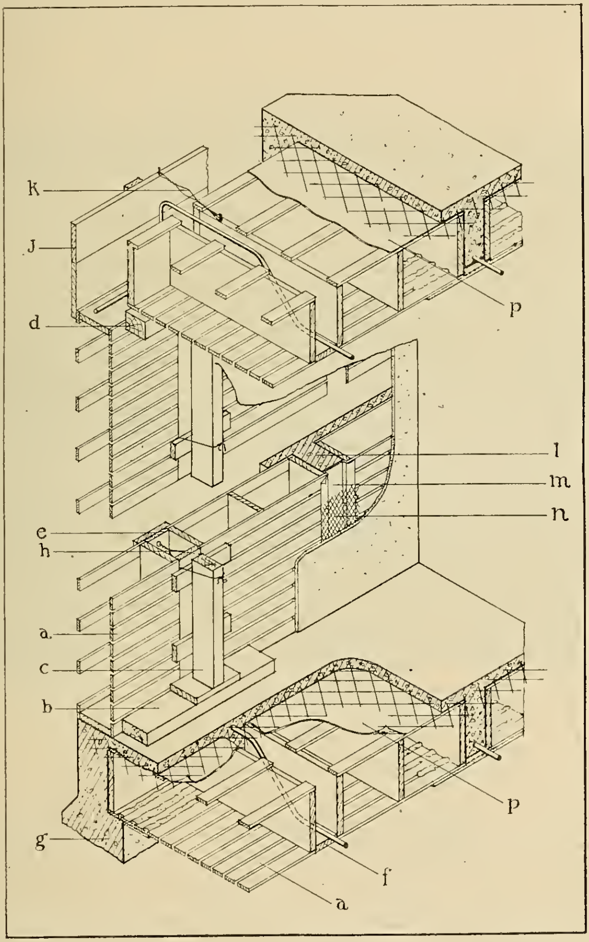


FIG. 2.—ISOMETRIC VIEW OF STRUCTURAL DETAILS.

The fireplaces are finished with a brick facing built against the concrete. The chimney is concrete, made with terra cotta flue lining as a core, and "Metaforms" for the outside form. A wooden buck forms the fireplace openings.

As the house is practically air tight, hot air heating is used. During the recent cold weather, there was no difficulty in heating every room.

The principal element of this system is the permanent wooden panel (*a*, Fig. 2 and 3), made of 1 x 6-in. or 1 x 8-inch boards, which has wooden laths nailed on one side and a backing or form (*p*) for gunite or concrete on the other side. During construction these panels were held in place by a temporary frame which is made of wood, and consists of sills (*b*), studs (*c*), plates (*d*), and outside boards (*e*) for the walls, and beam

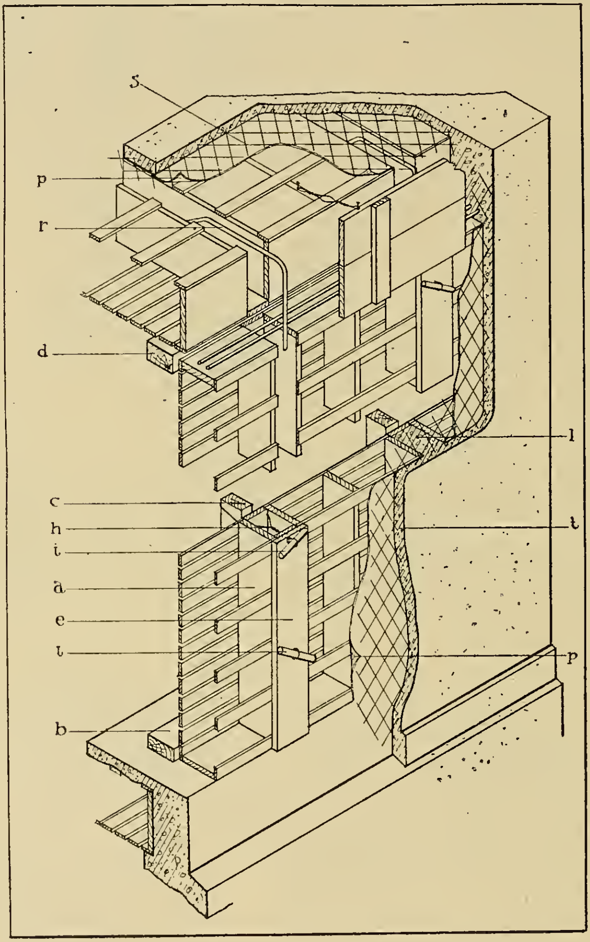


FIG. 3.—ISOMETRIC VIEW OF FORM LAYOUT.

bottoms (*f*) for the floors. The permanent panels are placed between the studs or the beam bottoms, leaving a space for steel (*r*), and concrete.

When the span is long, an extra line of sills, studs and plates is necessary to prevent excess deflection of the panels under the weight of the green concrete.

The method of erection is as follows:

The floor panels are first placed on top of the foundation wall, as shown in Fig. 2. They are supported at the other end by the forms for the central girder. The 1 in. board (*f*) which forms the beam bottom, is supported by the temporary plates (*d*), and by temporary battens nailed to the panels. A form is placed around the exterior of the building for the water table and concrete sill. Reinforcing is put in the beams and sufficient concrete poured to fill them. The slab reinforcing (*s*) is then put in place and the slab poured. The slab may be of ordinary concrete or nailing concrete, depending upon whether the finished floor is to be linoleum or wood.

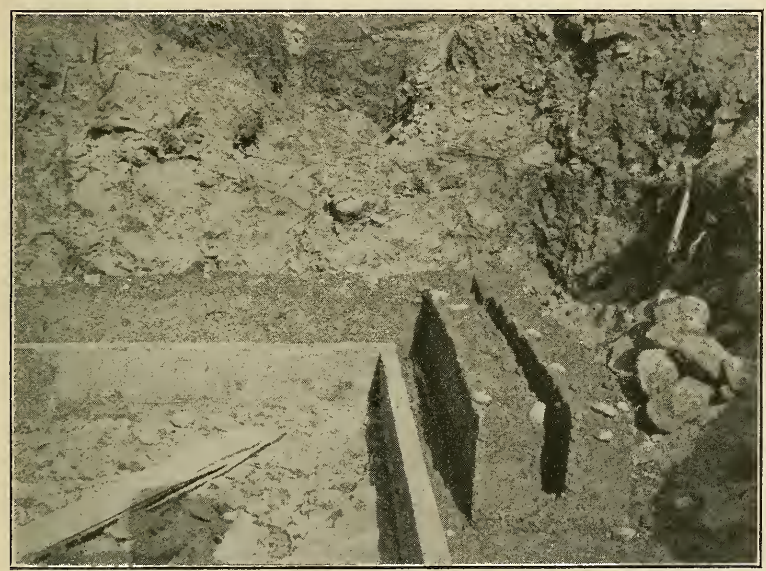


FIG. 4.—BEGINNING OF FOUNDATION WALL, USING "UNIVERSAL" HOLLOW WALL MOLDS.

These molds were used up to the first floor.

As soon as this concrete has sufficiently set to walk upon, the first story is started. First the temporary sill (*b*) is placed around the edge of the building on the line of the interior lathing. The studs (*c*) are then put in place; the cap (d) is put on top of the studs, and forms the line for the ceiling laths. This temporary frame is trued and braced. Loops of wire (h) are attached to the temporary studs as shown in the drawing.

In erecting the panels it is best to start at the corners and place the two corner panels in position, and hold them with two outside boards which have been nailed together at right angles. The loops of wire are passed through the outside boards and sticks inserted in the loops. When the sticks are twisted the outside board is drawn against the panels, making a tight fit.

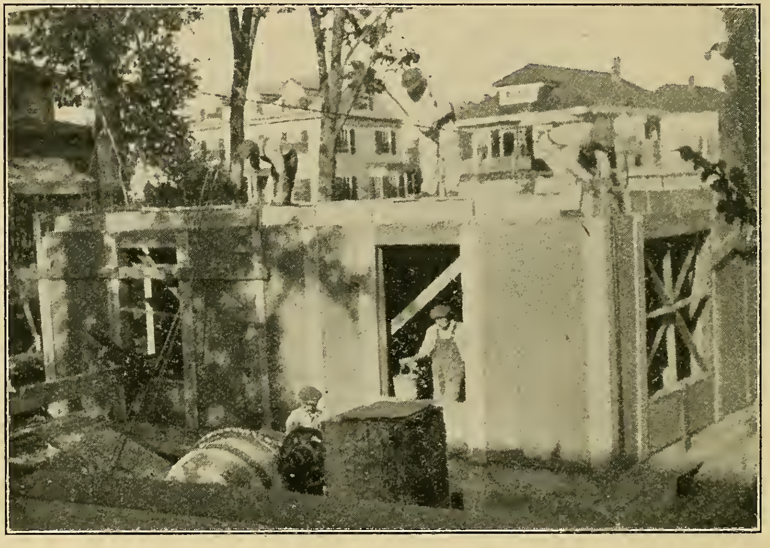


FIG. 5.—FIRST STORY PANELS IN PLACE READY FOR CONCRETE IN THE STUDS.

The form for the girt at the second floor has not been placed all around the building.

Proceeding along either side, the outside board is put against the other side of the corner panel, the loops of wire put through the holes in this board, and the sticks (i) inserted in the loops. The next panel is then slipped in between the temporary stud and the outside board. The sticks are then twisted, drawing the outside board tight against the panel. The process is continued around the building until all the panels have been put in place.

The second floor panels are now placed in the same manner as the first floor panels. The bottom of the form (j), for the girt around the second floor, is nailed to the top of the first story panels; the top is wired to the second floor panels (k).

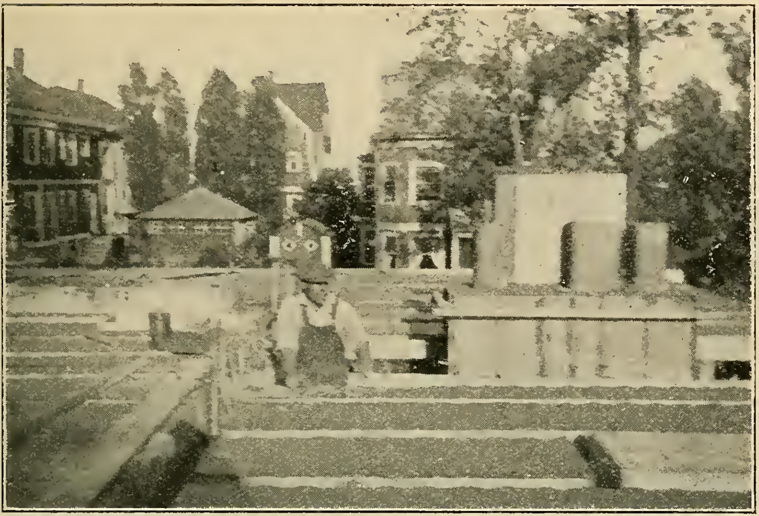


FIG. 6.—SECOND FLOOR PANELS IN PLACE AND A PART OF CONCRETE CHIMNEY.

The wooden buck for the rough fireplace may be seen just back of the flue linings.

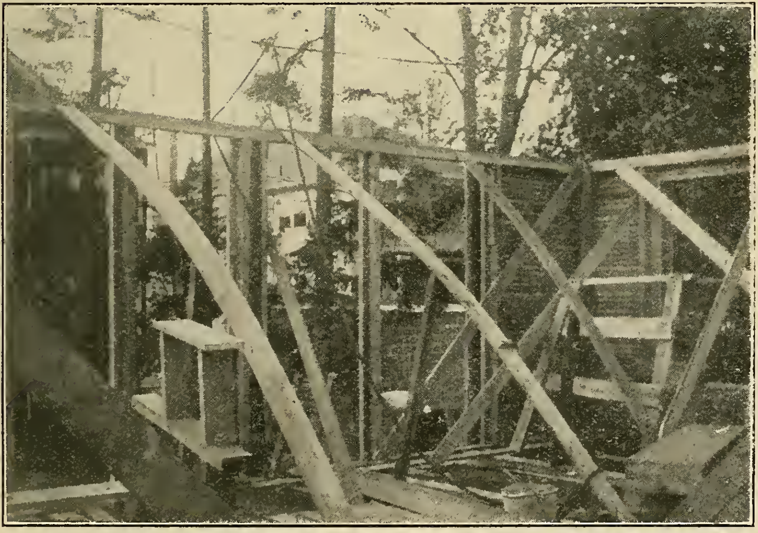


FIG. 7.—SECOND STORY TEMPORARY FRAME AND CORNER PANELS IN PLACE.

The studs (*i*) are now ready to be filled with concrete. After the concrete in the studs has settled sufficiently, the second floor is poured in the same manner as the first floor.

Succeeding stories are constructed in a similar manner.

After the concrete has set, the sticks are removed from the loops of wire and the outside boards pulled off. The inside of the loops of wire are cut and the temporary frame removed. The reinforcing mesh for the exterior is placed in position and the loops of wire bent up to hold this reinforcing in place. When the gunite (*t*) is applied a perfect bond is obtained between the gunite and the poured concrete, thus increasing the tie between the reinforcing and the concrete frame.

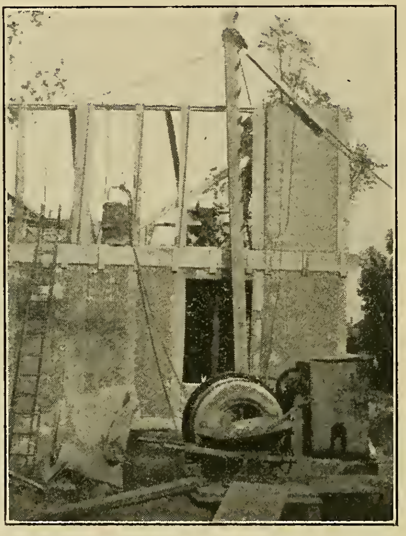


FIG. 8.—EXTERIOR VIEW, SHOWING SECOND-STORY PANEL PARTLY IN PLACE.

Before plastering, a heavy tarred paper (*m*) is put in the recess of the studs and strips of metal lath (*n*) are nailed to the panels on either side.

In constructing the Watertown house the floor and wall panels were covered with waterproof burlap. This material worked pretty well as a support for the 2-in. floor slab. There is some sagging between the cross pieces, which support the burlap, but this sagging increases the effective depth, and consequently the strength of the slab. Some difficulty, of course, was encountered in pouring the floor, because care must be exercised by the workmen in order to avoid stepping upon the burlap between the supports. However, the few accidents which occurred were easily remedied by covering the holes with another piece of burlap.

The photographs give an idea of the progress of the construction of this house.

Costs of House.

The direct cost, not including overhead or other indirect charges, of the parts of this house which are unusual was as follows:

1. Foundation and cellar wall.	
56 cu. yd. @ $18.00	$1,007.35
2. Floors, not including wood or linoleum top floor.	
3762 sq. ft. @ 0.498	1,874.93
3. Exterior wall, not including plastering.	
3177 sq. ft. @ 0.585	1,858.98

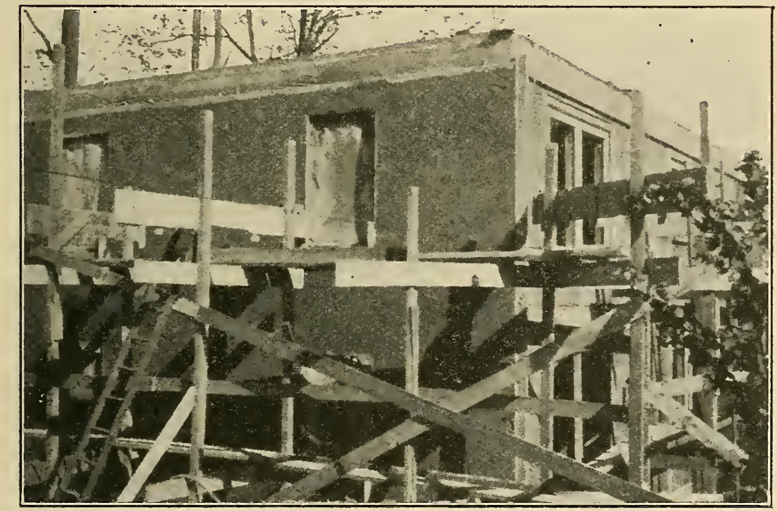


Fig. 9.—Building with poured concrete in place and gunite on one side.

4. Roof, including plates, rafters, etc.	
16.42 squares @ $38.20	628.10
5. Chimney	245.95
a. Fireplaces @ $42.25 ea.	84.50
6. Interior partitions, without plaster.	
2981 sq. ft. @ .195	581.35
7. Plastering.	
1318 sq. yd @ .593	781.20
8. Wooden Top Floors.	
975 sq. ft. @ .204	199.00
9. Linoleum Top Floors.	
127 sq. yd. @ 2.66	338.20

DISCUSSION.

Mr. Allen. LESLIE H. ALLEN.—May I ask Mr. Brown what means he has of insuring that the members of his structural frame are filled with concrete? He does not remove the forms. It seems to me they are such narrow members that he would have a stud that was not complete. Is there any precaution taken of insuring that the forms are filled?

Mr. Brown. H. W. BROWN.—The outer floors and inner floors both come up so we can see the stud the entire length; it is only the sides that stay in place. The panels are the only part of the structure that stay in place.

Mr. Allen. MR. ALLEN.—Does not that also apply to beam bottoms?

Mr. Brown. MR. BROWN.—No, beam bottoms come out, too. We can inspect every part of the poured concrete.

Mr. Havlik. R. F. HAVLIK.—What do you do with the floor forms? Do you remove them or do they remain in place? If they remain in place, how is that center wood joist held in place?

Mr. Brown. MR. BROWN.—The panel stays in place. It is made of three pieces or more in the case of some of the wall forms, with a board across either end that makes the frame comparatively rigid. That whole thing stays in place.

Mr. Havlik. MR. HAVLIK.—Does the center joist of the floor remain in place?

Mr. Brown. MR. BROWN.—Yes, that is a support for the laths; the wood plays a triple role; first, it is the form for the beam, second for the floor, and third a support for the plaster. It is 2 in. lumber.

MONOLITHIC CONCRETE HOUSE CONSTRUCTION AT PHILLIPSBURG, N. J.

By Paul R. Smith.*

In 1918, when the Ingersoll Rand Co., of Phillipsburg, N. J., wished to construct houses for employees, little thought had been given as to the kind of house which would be erected. It was necessary, first to build houses rapidly, economically, to make them attractive and comfortable and suit the needs of the various classes of people for whom the houses were intended.

A company was formed as a subsidiary company to the Ingersoll-Rand Co., known as the Phillipsburg Development Corporation. This corporation was to attend to all the details of buying suitable land, building roads, sidewalks, sewers, etc., designing houses, and erecting the same. Many conferences were held in reference to the best type of house construction for employees, many of whom would be foreigners. Two years before, the Ingersoll-Rand Co. had erected some one hundred frame houses near their plant in Painted Post, N. Y. This work was done with a great degree of success and the manager at Painted Post insisted that frame houses were the best type for the use to which the Ingersoll-Rand Co. wished to put the houses at at Phillipsburg.

At this time, there appeared in newspapers, and some magazines, photographs and descriptions of houses which were built at Union, N. J., by the New Jersey Materials Co. using wooden forms for concrete houses. These forms had been patented and manufactured by Charles H. Ingersoll of Ingersoll watch fame.

The most attractive element entering into houses of this character at that time, was the cost of such houses. After visiting the work done at Union and comparing the cost of such work as given at that time with frame house construction, it was finally decided that of the one hundred houses intended to be erected at Phillipsburg, seventy-five of these would be concrete and the Ingersoll-forms would be used. One of the reasons the concrete house was decided upon, was because of the fact that cement industries were close by and we would be able to build the houses with a large proportion of the cost, a common labor cost. It was further decided that due to the necessity for quick results, twenty-five frame houses should be built immediately.

Arrangements were made with a "ready-cut" house concern to furnish materials according to plans and specifications drawn by the corporation. Further arrangements were made with Mr. Ingersoll to ship the one set of forms which he had at that time, to Phillipsburg so that the erection of the concrete houses could be started simultaneously with the frame

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*Architect and Town Planner, Phillipsburg, N. J.

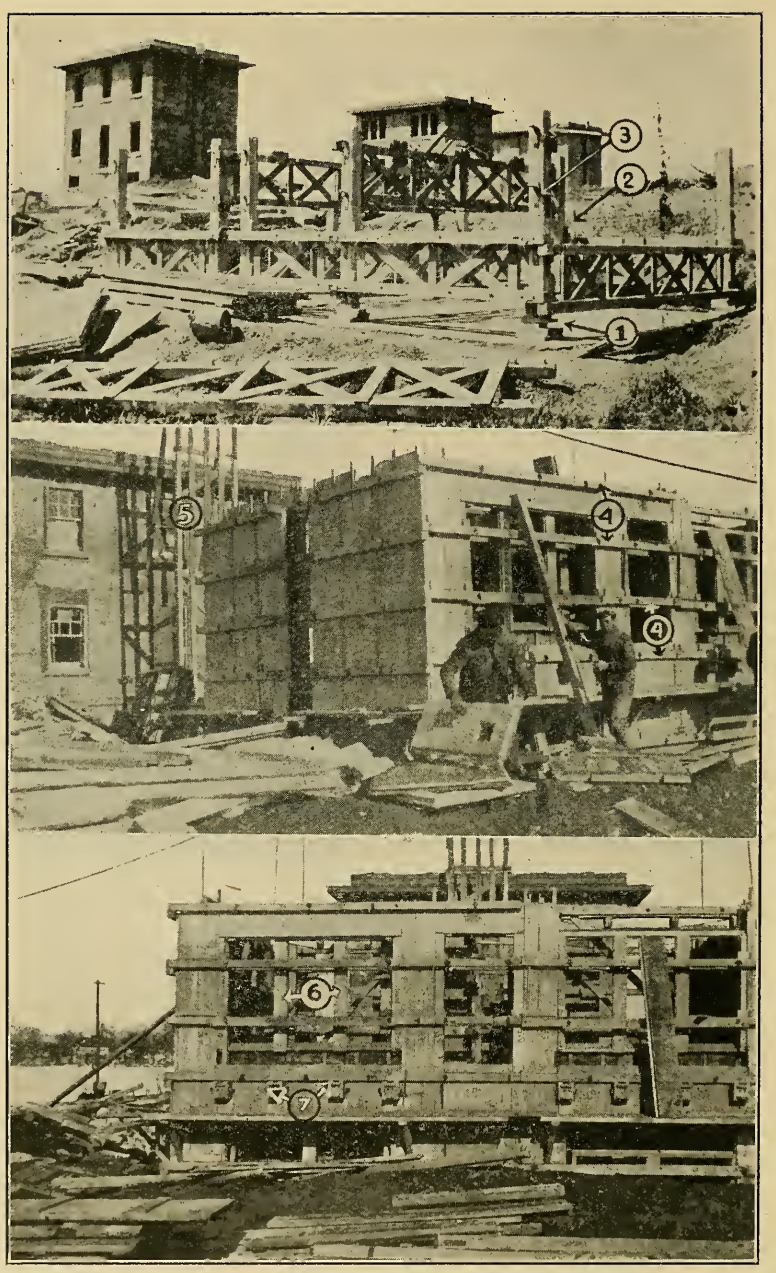

FIG. 1, 2 AND 3.—PROGRESS IN CONSTRUCTION OF 4-ROOM MONOLITHIC CONCRETE HOUSES AT PHILLIPSBURG.

(1) Corner posts set up on wedge blocks; (2) Truss joint with post; (3) Brackets on which trusses hang; (4) Outside 4 x 4 strings to align form sections; (5) Framework on which skip was raised in placing concrete in earlier work; (6) Window frames in place; (7) Forms for flower box brackets.

houses. Plans were prepared for a larger concrete house than the original Ingersoll house—the new house to have six rooms and a bathroom.

The parts and operation of the form system used is outlined as follows:

The Ingersoll system is probably the nearest approach which has been made to the theory of house construction advanced about ten or twelve years ago by Thomas A. Edison. The outstanding difference is the fact that in the Ingersoll system no top form is used for the floors. The hydrostatic pressure of the concrete in the walls, the pouring being continuous for the complete structure, is disregarded. Advantage is taken of the fact that concrete of a proper consistency allowed a few minutes in which to congeal, ceases to flow even under considerable pressure later applied. Pouring is first done on one side of the house till the concrete flows to proper floor level and then pouring is done on the other side, returning later to the first position.

The wood forms are built out of 2 in. stock crossed at right angles with best grade ⅞ in. lumber, creosoted to preserve them and thoroughly painted. They are oiled before use and cleaned after use to such an extent as may be necessary and oiled where the surface requires it. The excavation is first made: 2 x 4 in. side rails are staked down; footings and floor placed in one piece. From this basement floor, vertical corner posts 6 x 6 in., are first set up on wedge blocks, the posts extending to a story height. These are held plumb and true by trusses made of 4 x 4's and ⅞ in. bridging. Adjustment to true lines is virtually automatic. All parts are carefully made and permit no variation greater than $\frac{1}{16}$ in. The trusses are held to posts by iron bands made on each part and driven rigidly home by wedges. The form members proper, inside and outside, are supported by the trusses with bolts passing through from outside to inside with wooden washers for spacers, these being covered at each end by round iron washers strung on over the wooden washers to leave the round piece of wood well inside the wall when forms are removed. Outside forms are held further to line by horizontal 4 x 4 in. strings. The floor forms inside are also supported by the trusses, as also is the entire weight of the false structure and of the concrete as placed. Trusses carry the load to the 6 x 6 in. corner posts, ten of which are used in each story. The outer forms support brackets for staging, from which the upper forms are handled in erection and in stripping. The cornice and parapet forms are held by braces to outer forms, each piece having its precise location. In fact, every piece used has its unit number and a definite place in the erection scheme. Even with common labor, after a few operations, it has been possible to take a piece down from one house which has been poured, and set it in the proper position on the next house without rehandling. (At Phillipsburg a runway between two houses made this shifting very rapid.) When a certain stage of erection is reached the plumber and electrician are sent for and the conduits which are pre-fabricated are set in the walls. Reinforcing rods and metal flue linings are then placed and nailing blocks are set. Blocks are placed ready to hand for embedding in floors when they are screeded. The concrete used is one part cement, two parts sand, and four parts cinders.

In the erection of the forms the entire weight of the structure, both forms and concrete, is first placed on the corner posts, which in the four room house are ten in number. Yet these corner posts are first to come down when the forms are stripped. This is possible in view of the fact that the bolts passing through the walls and trusses, which support the forms, serve as a means of transferring the load to the concrete. As soon as a house is from 24 to 48 hours old under favorable weather conditions, the wedges are driven out, the corner posts are removed and set up for the next house in the operation. Trusses are then taken down and if the floor requires further support temporary shoring of about two 4 x 4's in each room are put in. The trusses are then removed and set up on the corner posts for the next operation. The form parts then come down in pieces as needed on the next job. The floor forms, which are usually the last to come down, are the last to be required to be set up on the next operation.

Due to uncertain labor conditions and to delayed shipments, the forms for the six-room concrete house did not reach Phillipsburg until the spring of 1919. At that time, the twenty-five frame houses started in November of the previous year, were practically all completed and many of the houses had been rented, while only ten four-room concrete houses were ready for occupancy.

At this time, it looked as though a serious mistake had been made in attempting to build the concrete houses at all. Possible renters would visit the ground, look at the concrete houses, be perfectly satisfied with their appearance and exclaim that the houses would surely be damp and cold and would immediately turn their attention to the frame houses. These they would rent until the houses were entirely filled. After the frame houses were all occupied, the people had to turn to the concrete houses. Although the four-room concrete house was very small, the people living in them were so perfectly satisfied and so surprised to learn that they were not only free from dampness but were extremely easy to heat, that many living in the frame houses wished to move to the concrete houses.

Then it was decided that all energies must be bent toward the completion of the concrete house program.

During the erection of the first few houses, it was rather difficult for the workmen to make any considerable amount of speed. This was due to the fact that the labor at that time was not particularly interested in working as there were too many jobs open to them. The laborers, however, soon got accustomed to handling the rather heavy wooden forms and under proper direction, considered the work as an interesting problem and from that time on, the erection of forms was speeded up. New ideas were suggested by some of the laborers which made their work easier and permitted the work to progress more rapidly.

After the first three houses were erected the work went on in a regular routine way, each man having his particular kind of work to attend to. The forms were so easily assembled that we were soon able to erect the forms complete and pour the concrete in a period of four working days (10 hrs.) with an average crew of twelve men.

After the cellar was excavated, the concrete floor was laid 6 in. wider than the total length and breadth of the building to be erected thereon. Suitable footings were placed at the same time that the cellar floor was laid. As soon as the concrete cellar floor had properly hardened the forms for the house were erected thereon. As the erection of the forms progressed, the window frames and door frames were set in place; reinforcing rods placed and tied in position; soil pipe and all other plumbing roughing necessary for the house was placed; electric light conduits were properly located and tied fast to the forms; the flue lining for the chimney was erected and the forms when entirely assembled, were then ready for the concrete.

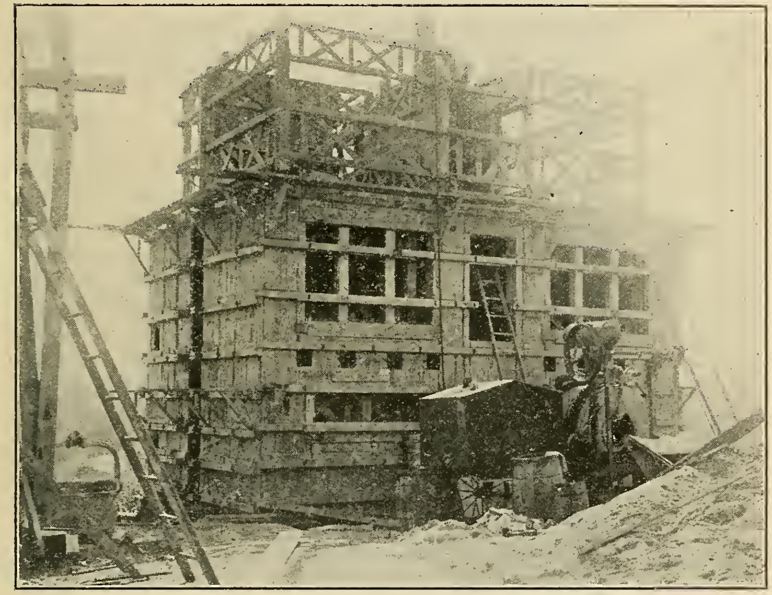

FIG. 4.—FORMS IN PLACE FOR BASEMENT AND FIRST STORY; POSTS AND TRUSSES IN PLACE TO SUPPORT SECOND STORY FORMS.

The concrete was poured from the roof. The conveyor was erected at the side of the building which carried a batch of concrete every minute and a half to a chute and hopper on the roof which in turn shot the concrete to the four corners of the house. The pouring of the concrete was continuous from the time the operation started until the last batch was placed on the roof. It was thought that there would be difficulty in getting the concrete to flow properly around window frames and door frames and across the floors. The forms are so arranged, however, that it requires little attention from the men who are located on the different floors as the concrete rises to keep the concrete flowing properly to its intended location. The concrete as it flows does not leave pockets around the window frames and door frames and due to the construction of the floor forms, the concrete would pass over the floor and down the partition forms and stop after it had reached the desired level, leaving the concrete floors true and ready to receive the grounds for furring.

It must be understood that the type of house developed is monolithic having concrete floors, partitions and roof and that the whole pouring operation involved only ten hours with twelve to fourteen men. The concrete was carried to the roof under power supplied by a gasoline hoist and the mixer used was a one batch gasoline "Dandy" mixer. The day following the pouring of the house, the work of taking down the forms may be begun if the weather conditions are suitable. During extremely cold weather it was practical for us to have heat in the buildings as soon as the concrete was placed and this heat was provided by salamanders placed in the basement and the first and second stories. We carried on the work during the winter of 1918-19 and 1919-20 without any loss of time unless it were due to storms. None of the concrete was frozen and the houses built during the winter are just as satisfactory as those built during the summer.

Most of the concrete used was cinder concrete. Stone concrete was used in a few of the houses and several houses were built with slag concrete. We found, however, that the cinder concrete was not only very practical and gave a sound and well constructed building but it was also very much more economical than either slag or stone. We found after a few houses had been poured with cinders and slag, that it was not advisable to use either cinder or slag for the roof. Stone was substituted when the concrete rose to the cornice so that the roofs were finished with a stone aggregate. The only difficulty encountered in using cinders was due to carelessness on the workman's part in permitting lumps of cinders to be mixed with the batch. These instances were few and caused very little difficulty. One of the most noticeable of these cinder pockets came in the partition between the bedroom and the bathroom where a cinder lump was retained on the plumbing roughing so that it left a pocket about 2 ft. square.

As soon as the second set of six-room house forms arrived, we were able, in good weather, to erect the shells of two houses per week. We had two gangs of men and were working both the six room forms and the four room house forms. From the experience gained with the use

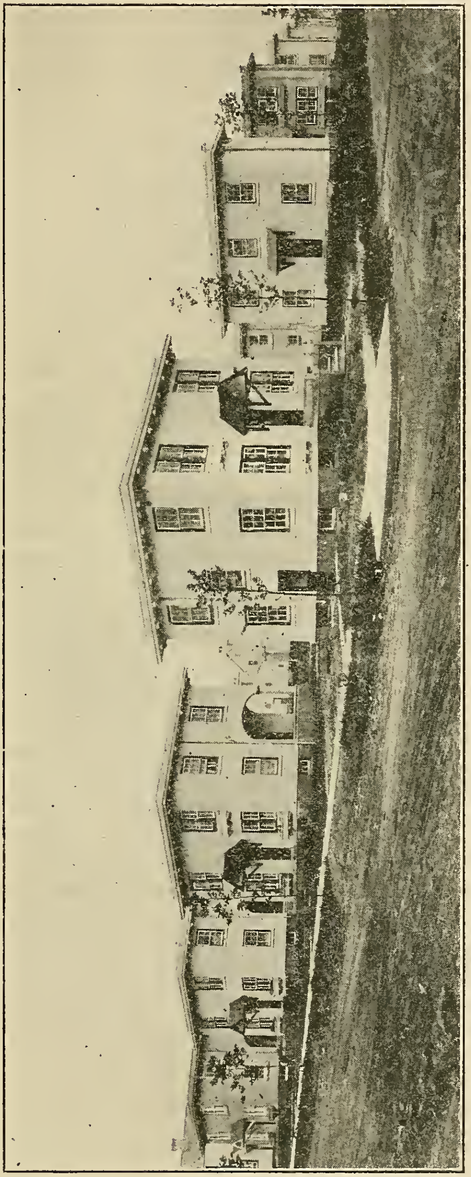

FIG. 5.—COMPLETE HOUSES OF THE 6-ROOM TYPE.

of the forms, we do not believe it would be necessary to make the forms in lighter units although lighter forms would undoubtedly cut the erection time materially and the labor would be less arduous. We had little complaint from the men in reference to the weight of the forms, their most repeated complaint being the handling of the forms during frosty weather in the early morning.

The amount of reinforcing that was used originally in the six-room concrete house was cut about twenty percent as the development went on. We found that this lessened amount of reinforcing was sufficient for all structural purposes. All walls are reinforced with ¾ in. and 1 in. rods carried vertically and all floor beams are reinforced with two reinforcing rods tied into the exterior walls and the partitions with cross reinforcing 12 in. apart. The cornice brackets and cornice itself are fully reinforced and tied in with the roof reinforcing and wall reinforcing.

As soon as the walls are thoroughly dried, the carpenters furr the exterior walls with 1 x 2 in. furring strips. On these furring strips Bishopric board is nailed to receive the plaster. The partitions are plastered directly on the concrete as is the ceiling in the first story. The second story ceiling is hung on the concrete beams supporting the roof and lathed and plastered. Wooden floors are laid on furring lath which is placed on the concrete floor inasmuch as it was not thought advisable to try to educate the people to concrete floors in this vicinity.

In speaking of the education of people, it is difficult for a person carrying on construction work through New York, New England, and the Middle West and Western states to appreciate the type of people whom we have had to contend with in our district. They were opposed to anything new and distinctly opposed to anything in the way of concrete construction. This opposition to concrete construction was due mainly to many houses having been built of concrete block in this vicinity in which the walls were left unfurred. The houses were damp, cold and unsatisfactory. They imagined therefore that the concrete houses which we built would be exactly the same. They had been accustomed to large kitchens and large rooms generally and were getting houses which they were satisfied with for a rental of ten to twelve dollars per month. To be sure these houses which they were getting so cheaply did not have modern improvements and most of them were without heat except that which could be furnished with kitchen or parlor ranges. When we asked a rental of $19.00 for the six-room concrete house, a prospective tenant thought that the price was extremely high. They did not consider the improvements which they had nor did they consider the fact that the house was detached with a minimum of 40 ft. on the street and 100 ft. in depth. It has taken two years for the people who have rented these houses to come to believe that they have an exceptionally fine house to live in. The complaints concerning the house are nil. The upkeep on the houses is extremely low, we having spent $20 per house only for painting the sash and frames. The houses are all supplied with pipeless heaters, electric light and running water. The pipeless heater keeps the house very warm and the amount of coal burned varies with the different families. We have had reports of families that have burned as low as four tons during the winter, which was extremely severe.

The changes made during the time in which we built the houses were minor. We changed the locations of electric light outlets which added to the practicability of the house. Instead of using black iron flue linings for the chimney, we incorporated concrete flues which were superior to the black iron. We omitted some of the reinforcing which showed a slight saving in cost with no harm to the building. The hoist used to guide the concrete conveyor was made an integral part of the forms and erected with the forms whereas formerly we had an unsatisfactory and unwieldy mast. We believe that it would be practical to dump the concrete in the first and second stories instead of carrying it to the roof and will use this method if any more houses of this character are built in the future. Undoubtedly the forms can be made lighter without affecting their longevity and at the same time decreasing the labor cost of erection. If the forms were made single ply instead of double or triple, the original cost would be lessened and the royalty on their use would undoubtedly be reduced.

The house itself has a square foot area of 20 ft. x 24 ft. 6 in. This includes the porch which is 7 x 12 ft. The kitchen, living room, and dining room are on the first floor and although small are very attractive and have plenty of light and air. The second story contains two good sized bedrooms with closets and a smaller bedroom without a closet and a bathroom having a concrete floor. The staircase leading from the first to the second story is of wood but a concrete staircase is made at the same time the rest of the house is poured, leading from the kitchen to the basement and to the side entrance door. The finish of all the houses is stucco applied with a brush. The color of the stucco varies with the different houses so as to overcome any monotonous effect of repeating the same building over and over again.

The costs on the houses (taken during 1920 which were the highest construction costs during the time that the building was going on) are tabulated herewith. These costs do not include overhead, royalty on the forms nor the cost of the lot which is placed at $70.00. Both our labor and material costs were at their peak during this period and we had excessive hauling charges as will be noted in going through the figures.

We are sincere advocates of the concrete house and are particularly well pleased with the monolithic type which we have used even though all houses are exactly alike and they have flat roofs. The Ingersoll-Rand Co. has shown its satisfaction by stating that no more frame houses would be built by them on the development. It would not be difficult or very expensive to construct the forms for these monolithic houses in such a manner that different elevations could be procured by interchangeable sections of form. Although we do not admit the necessity of doing this, in order to get an attractive appearance and a pleasing group of houses, there are undoubtedly many who would favor such a step.

TABULATION OF COST OF MATERIAL AND LABOR AT PERIOD OF PEAK PRICES

	Quantity.	Hauling.		Material.	Hrs.	Labor.	Total.
		Hrs.	Amt.				
Areaway		..	....	$1.20	2	$1.40	$2.40
Bishopric board	10 rolls	2	$0.85	35.00	18½	14.80	50.65
Chimney		..	....	9.00	2	1.20	10.20
Clothes post	2 iron	..	....	2.70	1½	1.27	3.97
Electric	Contract fixtures and hang	..	....	66.66	12	7.25†	73.91
Erection (forms)	469 spreader blocks	..	....	9.06	612	358.72	367.78
Excavation		..	....	.....	216½	97.88	97.88
Floors	800 ft. B.M	4	2.10	56.41	40	24.90	83.41
Foundation (cellar floor and footings)		..	....	50.42	55	33.09	83.51
Furring	1600 lin. ft	..	....	4.64	45	27.00	31.64
Grading		..	....	.....	93½	44.82	44.82
Heating	Pipeless base, registers	3	1.35	126.27	19½	14.09	141.71
Hoods		..	....	26.55	3	2.55	29.10
Leaders	Contract	..	....	7.80	..	.....	7.80
Nails		..	....	2.78	..	.....	2.78
Paint, inside	Contract	..	....	14.70	..	26.00	40.70
Paint, outside	Contract	..	....	6.21	..	25.35	31.56
Plaster	25 bags lime 32 bags plaster 1½ bbl. plaster paris 1½ yd. sand	15	7.30	49.15	con.	90.00	146.45
Plaster board	445 sq. ft	5½	2.35	17.00	12	9.60	28.95
Plumbing	Bath tub Bowl and tank Lavatory	7	4.15	167.74	con.	50.20	222.09
Painting and chipping		..	....	.....	53	27.64	27.64
Porch		..	....	3.62	31	15.60	19.22
Pouring	400 bags cement 45 yd. sand 45 yd. sand	127	66.94	578.03	180	103.37	748.34
Reinforcing	2000 lb	..	....	78.75	..	.....*	78.75
Sewer		..	....	6.57	68½	28.52	35.09
Sidewalk		..	....	7.33	10	3.44	10.77
Stairs, cellar		..	....	5.99	18½	12.55	18.54
Stucco	14 bags stucco 2 bags cement ½ bag lime	..	....	26.45	con.	50.00	76.45
Trim inside	12 doors 1 dresser Hardware 5 cellar sash, 18 pairs sash 1 stairway Base, jambs, casing Molding, etc	18	9.96	51.50 27.00 21.50 86.73 36.89 ..... 99.85	con.	90.00	423.43
Water line	Material in plumbing	..	....	.....	17	7.51	7.51
Waterproofing	45 gals	..	....	27.70	14½	7.25	34.95
Window and door frames	10 single 4 mullion 3 door 5 cellar	1	.42	108.92	..*	.....	109.34
Wrecking		..	....	.....	301	174.20	174.20
			$95.42	$1,820.12		$1,350.20	$3,265.74

* Labor for placing included in erection of forms.

† Rough.

NOTE.—Included in the above are clothes post, grading, sewer and sidewalks amounting to $94.65, which does not enter in cost of house. This price does not include overhead, cost of lot, nor royalty charge.

CONCRETE TILE IN HOUSE CONSTRUCTION

By Barton E. Brooke.*

As architect for the Youngstown Sheet & Tube Company since 1917 I have had experience with two distinct types of concrete houses, the unit system and the concrete tile house.

The one constructed under the so-called unit system of precast concrete slabs, a development in East Youngstown, Ohio, comprising 281 houses. This work was started in the fall of 1917 and completed in the spring of 1920. It received wide publicity through the magazines and the contractors, the Unit Construction Co., of St. Louis, I do not intend to go into detail on their construction. However, it might be of interest to know that these houses have stood the test in every way, the only exception being the appearance of condensation on the concrete walls when first occupied after vacancy during a cold period. The outside walls of these houses were cast with an exterior face of finished concrete, and with 1x2 in. creosoted wood furring on the concrete ribs in the interior. On this wood lath and plaster were applied. Although it was felt necessary to use this form of construction on the outer walls, it was considered safe to construct the inside partitions of hollow concrete units with finished face of concrete both sides. These walls have given the trouble. Over periods as long as two weeks after occupancy, moisture has condensed on them, while no condensation has ever occurred on the outside walls. From this experience and others, I strongly advise, that for rental houses at least, where vacancy is bound to occur, it is unsafe to use exposed concrete slabs or to plaster direct on any type of masonry walls. In every other respect, however, these houses are highly successful. No sign of depreciation of the structure is apparent and they receive the hardest kind of usage from foreign-born tenants.

At the time work was started on the unit concrete houses in East Youngstown, we started the development of a coal mining town at Nemacolin, Pa., for the Buckeye Coal Co., a subsidiary of the Youngstown Sheet & Tube Co., in which the houses were to be rented at maintenance cost, depreciation and interest being charged to the coal mined. They were modern in every respect, of five and six rooms, bath, and concreted cellar, with hot-air furnace, equal to the houses of well paid workmen in the cities, and not at all like the usual miner's house, which is of 4 rooms, hardly more than a shack, without basement, plumbing or heating, other than an open grate, and kitchen stove, and which rents for $5 or $6 per month. The rental of our houses was placed at $10 for the five-room and $12 for the 6-room type.

As rentals of this amount were double the usual rate, and houses of this type were new to the miner, many experienced coal mine operators laughed at what they considered the folly of the scheme. They predicted all manner of misuse of the conveniences, and failure to get miners to come to the town at the relatively high rent. This, however, did not prove a fact in either case. On the contrary, the company has never been without a large waiting list for houses from miners who wanted to work and live in the town, even during the war days, when every one was short of labor. Men came with their families and lived in camps for months, waiting their turn to get a house, as they were completed. With very few exceptions, plumbing and heating fixtures have been well cared for and greatly appreciated. This condition enables the employers to carefully pick their men, and today they feel that there is not a better manned mine in the country. The tonnage produced per man is far greater than in any other mine in the district, and the labor turnover and lost time is almost negligible. Through one strike in the district, and several decreases in wages, they have not had the least trouble, working continuously every day. I mention these facts, to emphasize that it is a paying investment to build modern houses, no matter what class of people are to occupy them, or to what they have formerly been accustomed.

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* Architect, Youngstown Sheet & Tube Co., Youngstown, Ohio.

Due to early transportation difficulties, these houses were planned with frame superstructure and monolothic concrete basement walls. Tonnage hauled to the town site was the most important factor of cost, as it had to be ferried across the river from the railroad and up a steep incline to the top of a hill at an elevation of 400 ft. above the river bed. Soon after starting work it was discovered that the average 5-room house as planned, would cost something over $8,000, and every effort was made to reduce that expense. The design of the house was simplified and the foundation walls changed to terra-cotta tile. These changes helped considerably but the tile foundations were homely, and by no means waterproof.

From the start I had been searching for some form of construction for the houses which would be economical under existing conditions, and at the same time give us strength, appearance, waterproof foundation and fireproof superstructure walls. Nothing was found until, while attending the National Conference on Concrete Housing in Chicago two years ago, I learned of the hollow concrete tile, which I felt would solve our problem. I therefore returned to Youngstown and designed houses for the use of tile in our future operations. This was not a difficult thing to do, as the units we intended to use could be made to fit any condition, and all that was necessary was to retrace the already accepted design for frame houses, simply increasing the thickness of the walls and changing the dimensions accordingly. After prevailing upon the company to purchase machines, and erect a plant to manufacture the tile, we selected our site, which was not an advantageous one, but the only available space in our railroad yard layout.

Erection of the plant started in April. Our engineers had worked out an excellent arrangement of the machinery, in a tall frame structure at the top of which large bins were constructed for sand and cement with batch measuring device at the hopper, which dumped directly into the mixer. The mixer in turn dumped directly into a hopper which connected through a chute to the tile machinery. The bins were supplied with sand and cement by bucket conveyors which together with mixer and tile machine were driven by electric motors. This arrangement was all that could be hoped for, but because of the fact that the plant site was on the uphill side of the railroad, it was necessary to erect a derrick with clamshell to unload cars of sand and place it at the foot of conveyors. Had it been possible to unload cars by gravity, this expense would have been eliminated. As it was necessary to have steam to cure the tile a line was run from the mine power plant and used to operate the derrick through a steam engine. Five kilns about 9 ft. wide and 50 ft. long were built adjacent to the plant, with walls constructed of the first tile produced and reinforced-concrete roof. Steam was piped to water sumps in each kiln. In these the newly made tile were placed and when full they were sealed so that the enclosed air was completely saturated when the steam was turned on. Under these conditions, the tile were left for 24 hours before being placed in the yard to dry.

An average of 2400 tile were produced on each shift, under favorable conditions, from one machine with nine men. With a four machine plant, it is possible under favorable conditions to produce, with an average of five men, the same number of tile from each machine.

We were in operation by June 1, but at first our production was slow, due to inexperience of the men. However, the product was excellent, and gradually the cost was reduced, until the tile were being manufactured at about the same price at which terra cotta of the same size could be purchased.

The tile were made with small corrugations on the outside face to receive stucco, and smooth on the inside. We had already discovered the bad results of plastering directly on masonry walls, and intended to use furring, wood lath and plaster on the interior.

By July 1, we had a sufficient number of tile made and cured to start construction, only to find difficulty in getting masons, even at $1.50 per hour and board. Mason contractors submitted prohibitive prices to lay the tile, due to their lack of knowledge of them, we were forced to abandon our ideas of fireproof structures, returning to the frame with foundations, and cellar walls of concrete tile. A total of 60 two-family houses of this type were completed during 1920 and 1921, together with an amusement building.

This building is the only one to date built entirely of concrete tile. The basement walls were constructed of 12 in. concrete tile, faced with 4 in. of brick and the superstructure of 12 in. tile and stucco. The labor costs on 12 in. tile walls was 15c per cu. ft., with masons at $1.50 per hour, and labor at 80c, and stucco was applied in two light coats, giving perfect results, at a cost of 50c per yd., as against $2 per yd., the price at the time for stucco on terra cotta tile. These are remarkably low costs for the time, and at least so far as the mason labor is concerned, should be cut in half today. I do not believe there is a building unit made, which can be handled and laid in a wall at lower cost than these tile, and I am sure that there is no base for portland cement stucco that compares with it in any way. The wall and the stucco, being of the same material, all difference in expansion and contraction which generally causes cracks in stucco, is eliminated, and the wall is so straight and plumb, that two light coats of stucco give perfect results, without the usual screeding and darbying required on more uneven surfaces. Hence a saving of both labor and material is made.

The house foundations were equally successful. They were covered with stucco and waterproofed below grade with perfect results. The walls were 8 in. in thickness and laid at a labor cost of 10c per sq. ft. of wall surface, and plastered at 50c per sq. yd. as was the theater. The completed foundation was amply strong for a two-story house, and constructed at one-half the cost of monolothic concrete, as well as giving a better appearance.

At the close of our operations last fall, we had manufactured a total of approximately 300,000 5x8x12-in. tile, and completed in all 275 homes, or about one-half of the ultimate town. I hope, for the next group, labor conditions will be such that we will be able to build all concrete tile houses, and continue to build them, for the balance of the town. The advantage of such a house is obvious in its comparative safety from fire, permanency and low maintenance expense. I believe that under favorable conditions of manufacture and labor it will not exceed a frame house in cost.

THE POSSIBILITIES AND LIMITATIONS OF PRESENT AND FUTURE MARKETS FOR CONCRETE BUILDING UNITS

By W. D. M. Allan.*

The United States, on an average, produces about seventy billion dollars worth of commodities of all kinds annually. Neglecting the deep inroads made into the natural stone industry in many sections by concrete dimension stone and further neglecting the hundreds of millions of concrete brick which are more than competing with clay both in the face and common brick field, and still further neglecting the rapid and satisfactory increase in the concrete lightweight building tile business and considering only the 8x8x16 in. concrete block which is by no means the most economical and satisfactory precast concrete unit, we find that out of every $2000 worth of commodities produced one dollar is represented by concrete block.

The industry deserves recognition. It is rapidly shaping itself into a formidable competitor in the permanent building material field.

Conservative estimates indicate that there are 6000 concrete products plants in the United States making concrete block. The annual output of these plants is well over 175,000,000. That 175,000,000 is a very conservative estimate, considering that the average output of the 6000 plants would have to be only a little more than 29,000 block annually. This is capacity of one hand tamp block machine operating eight months in the year.

The Portland Cement Association has been collecting data on the output of concrete block plants all over the country, and even though it is difficult to reach a large percentage of the plants yet it has been possible to show that many more block are used than was at first estimated.

I shall run the risk of being boresome by introducing a few figures; however, the figures presented will tend to show that the industry has made a healthy beginning. There are a number of products plants with annual outputs between 500,000 and 1,000,000 block per year and no small percentage of the total number of plants produce more than 100,000. E. Feltebaum, of Toledo, this year, sold between 800,000 and 1,000,000; Cunard-Lang, of Columbus, Ohio, 500,000; Omaha Cast Stone, 600,000; Ideal Cast Stone, Omaha, 750,000, in spite of the fact that the past year cannot be considered up to standard. Three-fourths of the plants here in Cleveland have output over 100,000 block, several producing 250,000 in normal times.

Figures collected during the last few weeks show a very satisfactory consumption of block.

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*Portland Cement Association, Chicago, Ill.

	During the Past Year
Cleveland	$4,000,000
Toledo	2,800,000
Columbus	2,400,000
Pittsburgh	2,200,000
Detroit	3,000,000
Davenport, Moline and Rock Island	2,220,000
Omaha	1,500,000
Providence	1,100,000
Minneapolis	3,000,000
Milwaukee	1,500,000
Calumet District, including Gary, Hammond and South Chicago	3,000,000

The markets named above in general are the leaders. However, there are a large number of other cities which compete closely, for example:

Springfield, Mass.
Miami, Fla.
New Kensington, Pa.
Buffalo, N. Y.
New York City
Camden, N. J.

From every source of information available we have found, and concrete products manufacturers agree, that about 90 per cent of all block go into foundations, the remaining 10 per cent go into garages, industrial buildings and residences above the foundation line. By far the smallest part of the output is consumed in residence walls above the first floor. In many cities 80 to 90 per cent of the residences built during the past ten years have block foundations. When people think of foundations they think of block. It has not required a selling organization or an advertising campaign to dispose of block for this use and with a few striking exceptions the block industry has been content with this class of work. The public has drifted into the belief that block are suited only for foundations. It is not uncommon to find block men ready to defend the stand that block are not suitable for residence construction. This is the course of least resistance, because it is not difficult to find architects, building departments and contractors who concur in this belief.

As long as the market for block is residence foundation work then the industry's grasp is practically the same as its reach for foundation work is assured. There are but few cities where block are not permitted for use in foundations and where some educational work must be done to change the attitude of building departments. Even this increase will not materially affect the business.

A brief study of the differences between the very successful plants operating under advershe conditions and the back yard, more or less unsuccessful ones operating under favorable conditions will develop the characteristic limitations of the business.

In spite of unfair competition within the industry and keen competition from the outside, the eminently successful plants have maintained a standard of quality which has built confidence and have used better merchandising methods. Block were sold on their merit; contractors were educated to see how they could make more money and get better results from block than from competing materials. In brief, their success was due to quality products efficiently merchandised.

Contrast this class with the one that cheated on quality and that bragged that it didn't have to sell its block because people came to his yard and got them as fast as they were made, that never increased its output; who met all competition with a cut in price and with a corresponding cut in quality; that rarely stayed in business more than two years. Thus it is that the "Crime of the '90's" became the "Crime of the 1920's." Thus it is that the building public condemns concrete block.

Inferior quality and poor business methods have done more to injure the business than all outside competition. The chief limitations of the business today are inferior quality, poor appearance, poor business methods and lack of confidence on the part of manufacturers themselves.

In 99 per cent of the markets in the United States, concrete block can successfully compete with other fire resisting and many combustible types of construction.

It is paradoxical that many of the best and most successful concrete block manufacturers are located in markets where the price of clay tile or brick is lowest. The following prices are taken from the "Brick and Clay Record" for Dec. 13, 1921:

City	*Brick*	*Building Tile 5x8x12*
Rochester, N. Y.	$16.50 per M	....
Cleveland, Ohio	14.00 " M	$50.00 " M
Columbus, Ohio	16.50 " M	....
Toledo, Ohio	15.00 " M	70.00 " M
Indianapolis, Ind.	16.50 " M	67.50 " M
Milwaukee, Wis.	15.00 " M	60.00 " M
Peoria, Wis.	14.50 " M	49.00 " M

The prices of clay brick and clay structural tile in this list of cities are considerably below the average. Nevertheless we have very successful plants in each of these markets.

Again it is paradoxical that where concrete aggregates and labor are cheapest it usually holds true that the block are most inferior. Two very striking examples come to mind.

First—Long Island, where one can find suitable aggregates anywhere, concrete block plants are so thick that no reasonably accurate record of them has ever been made. With very few exceptions, the quality of concrete block manufactured on Long Island outside of Brooklyn is very poor.

The second example is the State of Oklahoma. Suitable aggregates can be found anywhere and block plants are very numerous. Tulsa, a city of 80,000 inhabitants, has 19 plants all making block by hand. The Building Inspector of Oklahoma City recently stated that he did not know how many plants were in operation in his city. Norman, a small town of 5,000 people, had 5 plants. To give some idea of the general quality of the block made in Oklahoma, the following tests are given which were made a few days ago at one of our Home Builders' Conferences at Oklahoma State University.

A.	Concrete	block	.... 248 lb. per sq. in. of gross area
B.	"	"	.... 275 lb.
C.	"	"	.... 340 lb.
D.	"	"	.... 250 lb.
E.	"	brick	.... 800 lb.
F.	Clay	"	.......6250 lb.
G.	Benedict cast stone		....4175 lb. did not crush

It is a matter of common knowledge that concrete block failed to stand up in the fire in connection with the recent riot in Tulsa and this is largely due to poor aggregates, obsolete manufacturing methods and lack of enforced standards of quality.

If it were not for two very important changes that are coming about there would be less reason to be sanguine over the future of the business. First, more building codes are under revision at the present time than at any time in the past. In the state of Ohio alone about 12 codes are being brought up to date with regard to concrete block. Probably there are more than 100 codes in the United States that are being revised at the present time. Building commissioners are usually glad to make their codes conform with the American Concrete Institute specifications governing concrete building units. With the exception of a few places where the A. C. I. specifications are not up to standard, the newly revised codes very closely follow the recommendations of the Institute. For this reason there is a very urgent need for the American Concrete Institute to prepare, and thoroughly revise forward-looking specifications covering concrete block. By block is meant all precast unreinforced concrete building units. The American Concrete Institute specifications always should be kept one pace in advance of the time because as a general thing building codes are not revised frequently and their provisions are likely to become obsolete unless they are right up to the minute when the code is adopted. Many building commissioners hold to the theory that unless the code specifically permits a material under specific regulations that material is barred. However, the newer codes show a decided step in advance in that they permit any material or type of construction not specifically provided for in the code to be used as soon as the Building Department is satisfied that the material is structurally sound and meets the sanitary requirements of the city. In other words, the older idea is that a code once written is complete and is closed to anything not included in it; while the newer idea is that a code is never complete or closed and that as soon as a new material proves its worth to the satisfaction of the Building Department it shall be permitted. Again, that the code is to encourage satisfactory construction and not to hamper progress.

Not only is there encouragement from the city building code situation but from state and possibly national codes as well. Ohio and Wisconsin have state codes at present. West Virginia and Oklahoma and other states are considering the advisability of state building regulations. A state code is necessary for far more than half the building materials used in the country are used outside of cities large enough to have building codes with enforcement officers.

The Hoover committee which is at work on a standardized national code should do a great deal to eliminate wide differences at present existing in building regulations. We recently had a communication from this Committee stating that it did not have information enough available to warrant allowing concrete block for foundation construction. It was necessary for us to send out an S. O. S. to the leading block men urging them to tell us how many block foundations were in use in their cities. Out of 45 cities that replied the returns showed that nearly 128,000 residences had block foundations. Brooklyn, the only large city to make a return so far, showed that 50,000 buildings in the borough had block foundations.

Building codes which are strict and provided with teeth for enforcement which conform with the specifications of the American Concrete Institute hold out a positive method for the standardizing of quality of concrete block. With quality standardized one of the chief prejudices will be removed.

The second important change that is coming about is that a higher type of business man is being developed within the business itself and the new men starting into the business are of a higher type.

On every side one could see during the past year better business methods. Products manufacturers formed associations and actually put on co-operative advertising campaigns to educate the public to demand quality products. Block were trade marked and tests were made in reliable testing laboratories to see if the block met the requirements of the American Concrete Institute.

The products plant of the past consisted of a block machine, a mixing board and a few pallets, while today there are many plants with $75,000 to $100,000 invested, with automatic machinery and conveying equipment and every mechanical device to reduce labor costs.

Along with rapid developments in machinery and plant equipment there has been a decided change in the size and weight of units. The time was when nearly all concrete block were about 8x8x32 in., weighing approximately 120 lb., with thick webs and small air spaces. From this large, unwieldy block with small air spaces, there has been a constant reduction in size and weight until today probably the most efficient unit is the lightweight building tile which weighs approximately 20 lb. It is 5x8x12 in. for an 8- or 12-in. wall and replaces approximately five bricks. The webs are thin, and the air spaces large. The lightweight tile bids fair to become a widely used unit because it is saving in material and is very easily laid. Owing to the hand hold in the top and to its light weight, a mason can lay it without dropping his trowel. It is a one-handed unit and beds easily in the mortar. A mason can lay much more wall per day with the lightweight tile than with any other unit.

One of the chief obstacles in the way of the more extensive use of the 8x8x16 in. concrete block is the apathy of masons towards the heavy unit. The lightweight tile removes this disadvantage.

A few minutes ago we pointed out that concrete block were used almost exclusively for foundation construction and further that as long as the block were used for foundations alone, the market must remain limited to a small part of what is in reality a tremendous potential market, namely residence construction.

The lightweight tile opens the residence field because it is more economical than block and will compete with standard frame construction at an initial cost about 3% more than frame. The lightweight tile gets us entirely away from the rock face which caused all reputable architects and discriminating abortion builders to line up with the opposition. In residence work it will be nearly always necessary to use stucco.

Specifications governing concrete building units should take special cognizance of the lightweight tile, requiring that the quality of concrete in the tile be kept up to the same high standard for which the Institute stands, but not requiring a gross area test out of all proportion to the load which the tile will be required to stand.

As plants become larger and more complex and it is more difficult to start in business, the chances for the back-yard plants become poor. It is true that a large part of the successful plants in operation today started with hand tamp outfits, but conditions have changed so much that very few successful plants will start in that way again. While the demand for block and tile will increase rapidly relatively fewer plants with larger outfits will be the rule. Instead of 19 plants in Tulsa all making a poor quality of blocks, and all competing for the cheaper class of trade on a price basis alone, there will be two or three plants all of them turning out products of standard quality competing for the best class of trade on a service basis. The time draws near when the per capita consumption of concrete block will increase from five to ten times.

REINFORCED-CONCRETE FIREPROOF CONSTRUCTION APPLIED TO HOME BUILDING.

By P. J. Hueber.*

To be specific in discussing reinforced-concrete construction applied to home building, to bring out its architectural, engineering and commercial possibilities for any type of home, the same as for our larger buildings, let us begin by dividing building into three types, namely:

Type 1. Wood, which is costly to keep up, and easy to burn down. Any builder can erect a wood-frame dwelling, with the result that competition is keen in this type.

Type 2. Masonry walls with wood interiors, which saves on upkeep expense, but as 96% of the home fires originate from within, according to a New York City report, this type offers very little fire protection and here concrete competes with brick, tile, or natural stone.

Type 3. Masonry walls and masonry interiors. We are interested in concrete masonry which requires a minimum upkeep expense and offers a maximum fire protection. It is recognized as the best for hotels, school houses, office buildings, country homes, etc. It is in a class by itself. It has absolutely no competitor.

Why can we produce such wonderful results in large concrete jobs and why cannot we successfully use the same material in our homes, a number of which comprise a large job? And with a shortage of thousands of homes why think only in the unit of one? In other words, what are the basic principles for the successful use of concrete in one class, and how can we apply them to home building?

In order that we may have a mutual understanding of just what we mean by reinforced-concrete construction for the home, we shall go over the elements necessary in home building to determine the possible concrete parts.

The footings and cellar wall are usually of concrete in any type of construction. The exterior wall above grade in the usual reinforced-concrete job is, however, on the order of a brick and tile wall, the floor being carried on exterior columns, but in case of the home, by moving up our cellar wall forms, we can economically build the entire exterior wall of concrete, furring it later on. In a fireproof home the floor will naturally be of reinforced concrete, having any desired finish, and carried on interior columns and beams; while the non-bearing partitions will be of, say gypsum tile as in any fireproof building, since the use of poured concrete will necessarily cut up and complicate the exterior wall and floor forms.

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*Of Hueber Bros., Builders, Syracuse, N. Y.

and also hinder freedom in varying the room arrangement of a group of homes.

The roof construction will naturally be determined by the architecture, concrete if flat and slate covered frame construction if pitched roof is called for. The stairs can be safely made of wood, protected underneath by metal lath and plaster, or they can be of concrete. The remaining items for exterior and interior finish are practically the same as in any type of construction.

Sifted down, then, the use of concrete in home building is economically limited to the structural frame; the cost of which concrete frame will be the identical for two homes of same size even though one is in a wealthy section and the other in a poorer section. The structural frame is determined by the building code standards, while the finishing is determined by the desires and purses of the people for whom they are built. The structural frame of a factory will cost the same as one for a similar sized hotel (if the live-loads are the same) but the finish is another matter in which the concrete plays no part. As far as concrete is concerned, the problem is one of economically producing a structural concrete frame, so often considered as the house itself. Here we have a frame for a home that will not crack when properly reinforced, that fire cannot burn, that time cannot fade, that rain cannot penetrate, that winds cannot budge, and that vermin cannot disintegrate.

This concrete frame is naturally determined by the *forms* into which it is *poured* when in a plastic state. Think of the adapatability of concrete in this state of manufacture, as it comes from the mixer and for what more flexible material can your ask with which to build a frame for a home?

As the first thing in a development of a number of homes, no matter of what type, the builder demands standardization to cut his costs. The apparent horror of standardization seems to stop further architectural interest when a set of forms is mentioned for fireproof homes of reinforced concrete. But standardization is not a hindrance when we understand it but, on the contrary, it makes for efficiency, economy, and permits of good architecture when properly used, as we shall prove.

Investigate any of the successful homes of wood construction and you will find builders have cut their costs by erecting a well proportioned frame to a few simple plans and have obtained pleasing architectural results in finishing this frame with wood, brick or stucco, and by proper handling of roofs, porches, etc., all built up of standard units of standard shapes. Variety was further gained by different placing of standard windows and doors, and various pleasing color effects were obtained by applying standard paints. Later on the owner sows standard lawn seed and plants standard shrubs, and the same gentleman soon cuts the standard grass with a standard lawn mower, and only cusses the non-standardized weeds. So why worry about standards, for by merely combining them differently we get results that certainly do not look standardized. People like individuality in their homes and will readily pay for the small extra cost of obtaining different exterior designs as well as different arrangement of rooms of similar sized plans.

There certainly is no reason why we cannot also finish a concrete frame as we do a wood frame with various architectural results. So let us study this concrete frame which, as we have said before, is first a question of forms. What better can we do than to study the form construction used by successful concrete contractors all over the country? Their form for a job requiring concrete walls and floors consists primarily of a sill course, studs, plates and floor joists, all sheathed over and properly fastened and supported. Fundamentally, these forms are nothing other than our typical structural parts used in framing a wood house, without cutting for windows and doors.

It is very evident then that before we pour our concrete for a building, we must practically erect an ordinary wood frame for our structure as a form from which we will obtain a duplicate in concrete. Therefore, since it is apparently a matter of covering our present wood frame with a concrete frame and then removing the wood one, the study of the forms for so-called concrete house brings us back to the study of the structural elements of ordinary wooden house framing, to see how they can be adapted for standardized forms, for obtaining a duplicate frame of concrete of various sized plans.

You will find on examining popular plans of these wood homes of colonial days as well as of our present time that the vast majority are rectangular in shape and on analyzing them for structural members you will see that the smaller rectangular homes have one bearing partition, which naturally in a concrete job will be replaced by beams and columns. In making this type longer the joists on either side of the bearing partition must be longer up to a certain limit, and then a still longer home is obtained by adding a second bearing partition, as in the popular center hall type of home. To make either plan wider, one naturally calls for additional joists and studs. Various room arrangements are readily obtained by changing the interior partitions and stairs. Since people demand smaller or larger homes of each type, why not vary the sizes in terms of a standard unit say 40, instead of a few inches as it materially reduces the number of necessary plans to a few standard ones, a vitally important matter for forms. The architectural success of these wood homes cannot be questioned as there are thousands of examples to prove it a fact, and all we want to do it is duplicate the unseen structural wood frame with an unseen structural concrete frame; finishing it the same, if you wish with clapboards, stucco or brick, and a roof.

With good architecture thus assured, the whole question now then centers down to the vital question of costs. If we can build the wood frame (forms) in knockdown so as to eliminate the endless labor of measuring, sawing and nailing each piece of wood for each house, and at the same time obtain different sized plans without complicating these forms, which we must use over and over again, we have one method, at least, for economic production of concrete frames for fireproof homes.

Our solution—the Hueber Forms (patented)—consists of wood studs resting on a wood sill course with beveled top to allow easy stripping of studs. A steel channel plate (or aligner) carries around the building at top of studs an dalong the beam sides to tie and line up the studs properly, and it also carries the floor form joist of light pressed steel I-beams, which do not require shoring. Braces adjustable by turnbuckles and pinned into position plumb up the walls and horizontal angle iron braces at inside corners of building at aligners automatically square the forms. (The four corner studs are temporarily braced with short pieces of wood nailed to sill in erecting.) Wall panels are buttoned in place and floor panels are simply placed on joists. Exterior forms consist simply of one sized panels; while one sized wall tie and one sized wedge are used to hold the forms, a standardization which avoids confusion by eliminating the possibility of mixing up small pieces of different sizes on the job. Separators thru which malleable iron wall ties (bolts) pass are used in holding wall forms.

Different sized homes are obtained by simply adding or replacing a span of joists and plates (or aligners) paralled to joists, just as is done in regular framing for wood homes as we have before stated. We do not use adjustable form pieces as they hinder efficient field work. Sleeves for plumbing and heating pipes are placed before pouring, while the conduits for electric wires are placed in floors and exterior walls before pouring just as in any concrete job.

Our aim is to make possible the production of structural concrete frames of various sizes and adaptable to various architectural results, at a rock bottom cost. Thereafter, as we have stated before, what goes in to make up the finished home in plumbing, heating, electrical equipment, roof, porch, and decorating must rest with the builder architect or owner as to how much he thinks it advisable to spend in the particular locality. As to costs of the concrete frame, including erection of forms, placing of steel reinforcing and sleeves, pouring concrete and stripping and moving equipment, it takes 360 hours using 2 carpenters and 3 laborers, for a completed frame with 6 in. walls and 5 in. floors for a 24x31 ft. home, with 8000 sq. ft. of total form surface for walls and floors. About two tons of reinforcing are used.

By using sash balances in place of pulleys and weights, we can use a ½x2 in. casing—a saving of material and time in erecting. This trim is also used for doors and people like it. In a lower priced home than we are now building the tile and ⅞ in. oak floor finish can of course be replaced by linoleum, or concrete. Our interior partitions of gypsum tile can be replaced by a 2 in. metal lath and plaster wall, or wood studs with metal lath or plaster boards. Since there is no chance for settlement, the danger of cracking plaster is avoided, and we use this as a good selling point for in so many wood homes cracks are unavoidable and also every lath shows. We consider the money spent for furring our 6 in. walls with 1 in. corkboard nailed directly to wall and ready for plaster, is a good investment when yearly saving in coal is figured. Metal lath or plaster board may be used, of course, in place of cork, but in this case the necessary furring strips should be nailed on horizontally, to shorten the air spaces as the higher an air space is the less efficiency it has. In warmer climates furring is unnecessary in which case a thin plaster coat can be applied directly to wall. In either case it is the cost of a unit of a *finished* wall that must be considered in making comparative costs, regardless of how costly it appears to be. It is not necessary to furr for a brick veneered concrete wall as it has an air space between brick and concrete.

We use beam and tile (gypsum) slab construction for floor instead of solid slabs, not only saving on dead weight without increasing cost, but also we get a warmer second story through the better insulating properties of gypsum in the second story ceiling, or one might say on the attic floor. Framing around stair well, which can be placed as desired, is obtained by placing a 5 in. steel channel on forms as a beam and after stripping our form, we place a small sized precast concrete column under the channel. Breaks in plan for special demands can be obtained by using ordinary formswork along with our forms, using this lumber later on where possible, or when several homes with like breaks in plan are built, an extra unit of our forms can be built for it and used over for each home. Bay windows are more economically framed out with wood and covered with lath.

The cost of forms is immaterial as we intend only to lease our forms and the item of royalty will depend on individual requirements of architecture, etc., of different localities. While our firm, as general contractors, builds schools, factories, etc., of concrete construction at the same time we believe that in concrete construction in home building there is a large field open to the progressive builder.

NOTE ON FATIGUE OF MORTAR.

(From the Laboratory of Testing Materials of Purdue University).

By W. K. Hatt.*

Recently the need for information upon the ability of plain concrete to withstand repeated loads has become evident, not only in the design of road slabs but in the interpretation of tests made from concrete roads of different ages. Experiments in compression by Van Ornun, of Washington University, indicated the fatigue element. Since then experiments have been conducted in reinforced concrete by the United States Shipping Board.

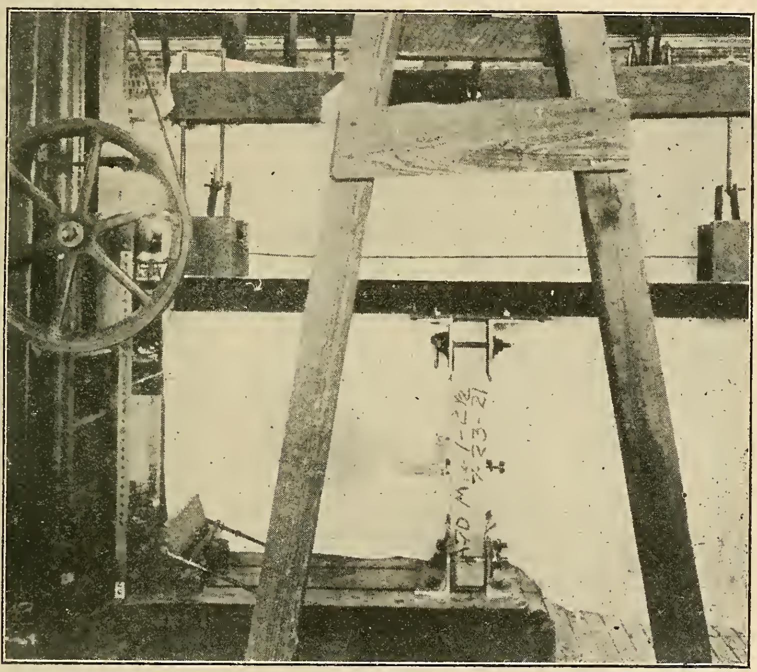

FIG. 1.—FATIGUE TESTING MACHINE IN LABORATORY.

At the present time three investigations of plain concrete under repeated and reversed stresses are under way in the testing laboratory of the Department of Highways, State of Illinois, under Clifford Older; in the testing laboratory of the University of Maryland, under Dean A. N.

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*Professor of Civil Engineering, Purdue University, Lafayette, Ind.

Johnson, and under the direction of the writer at Purdue University. It is the purpose of this paper to give an account of the experiments of Purdue University, not with the view of drawing any conclusions, but as a matter of information of current progress and to stimulate others to enter this fascinating field of research.

It is well known that metals will rupture under repeated application of loads below the ultimate strength, and that there is a percentage of the ultimate stress which may be applied indefinitely without rupture. It used to be considered that the law connecting stress and number of repetitions was a continuous curve such as shown by Van Ornun, but Prof. H. F. Moore has demonstrated quite clearly that there is no continuous law, and that at stresses higher than the critical stress the number of repetitions required to rupture diminish rapidly as the stress increases.

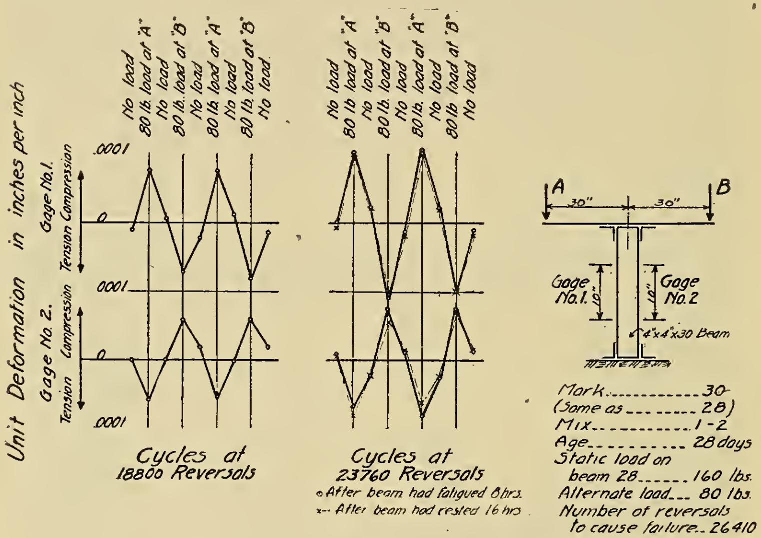


FIG. 2.—TYPICAL CYCLE CURVES OF FATIGUE TEST.

It might naturally be expected that a material like concrete which has only a small degree of elasticity and a large degree of plasticity, would exhibit phenomena of fatigue. The difficulties in measurement in the case of plain concrete are serious, and the effect of the inertia of the specimen likely to be confusing.

APPARATUS.

The apparatus used in the Purdue experiments is shown in Fig. 1. The specimen is a mortar beam 4 x 4 in. cross section and 30 in. long. It is placed in an upright position as a column and loaded by a bracket fixed

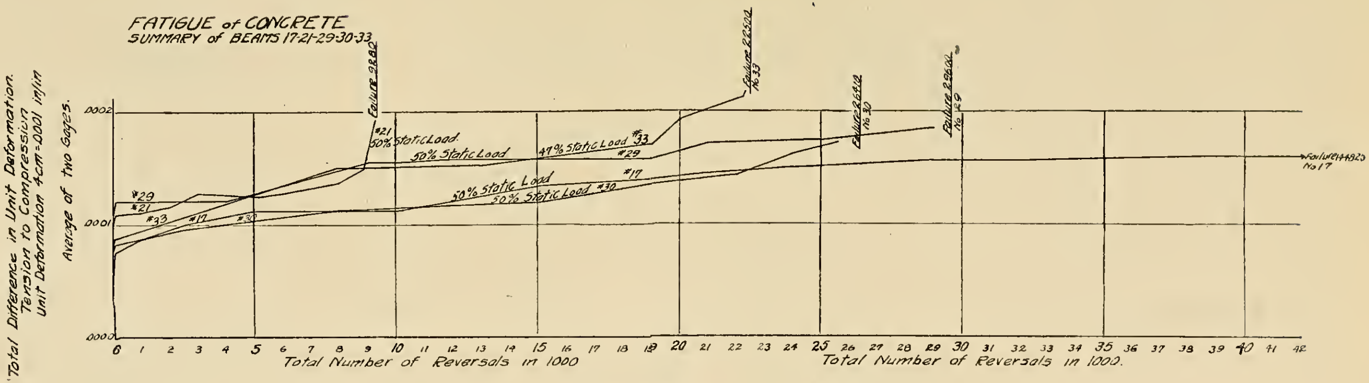


FIG. 3.—FULL CYCLE CURVE OF ONE BEAM.

TABLE I.—FATIGUE TEST.

CYCLE CURVE DATA.

Beam—No. 30.
Mix—1 : 2.
Age—28 days.
Alternate load—80 lb.
Per cent of Static Load—50.

18800 REVERSALS.

Load at	Unit Deformation, in. per in.	
	Gage No. 1.	Gage No. 2.
—	+.000 010 5	+.000 000 2
A	−.000 076 8	+.000 058 7
—	−.000 004 8	+.000 000 2
B	+.000 071 8	−.000 058 3
—	+.000 022 0	−.000 017 8
A	−.000 075 4	+.000 057 6
—	−.000 011 5	+.000 002 7
B	+.000 080 4	−.000 058 1
—	+.000 013 5	−.000 017 8

NOTE. + sign is tension, − sign is compression.

23760 REVERSALS (after beam had fatigued 8 hours).

Load at	Unit Deformation, in. per in.	
	Gage No. 1.	Gage No. 2.
—	−.000 000 2	−.000 008 7
A	−.000 103 0	+.000 067 6
—	−.000 022 8	+.000 022 7
B	+.000 109 1	−.000 074 4
—	+.000 012 7	−.000 010 3
A	−.000 105 8	+.000 081 2
—	−.000 022 0	+.000 025 8
B	+.000 099 6	−.000 074 6
—	+.000 010 5	−.000 012 4

NOTE. + sign is tension, − sign is compression.

23760 REVERSALS (after beam had rested 16 hours).

Load at	Unit Deformation, in. per in.	
	Grade No. 1.	Grade No. 2.
—	+.000 009 1	−.000 008 7
A	−.000 102 2	+.000 067 6
—	−.000 020 8	+.000 022 7
B	+.000 100 2	−.000 074 4
—	+.000 019 6	−.000 010 3
A	−.000 098 6	+.000 081 2
—	−.000 019 8	+.000 025 8
B	+.000 100 0	−.000 074 6
—	+.000 019 6	−.000 012 4

NOTE. + sign is tension, − sign is compression.

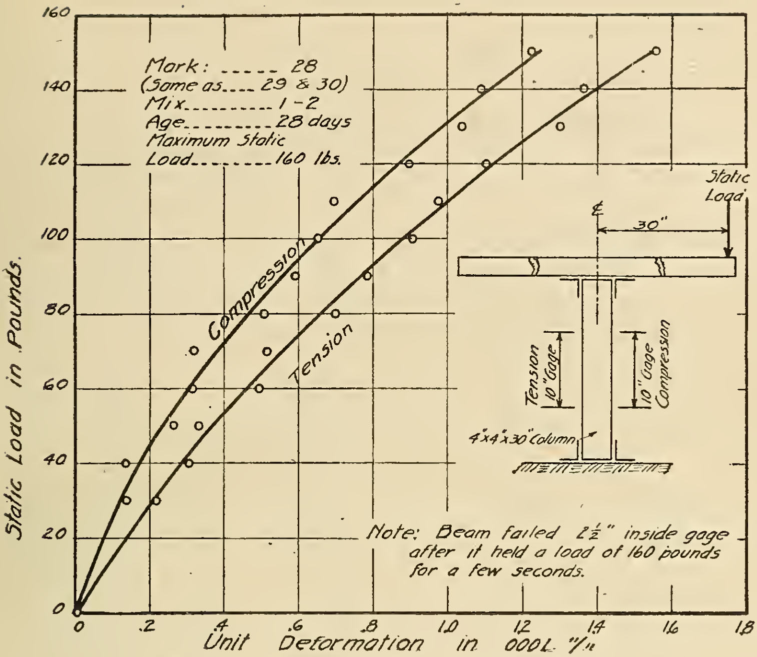


FIG. 4.—STATIC LOADING CURVE. (See Table II.)

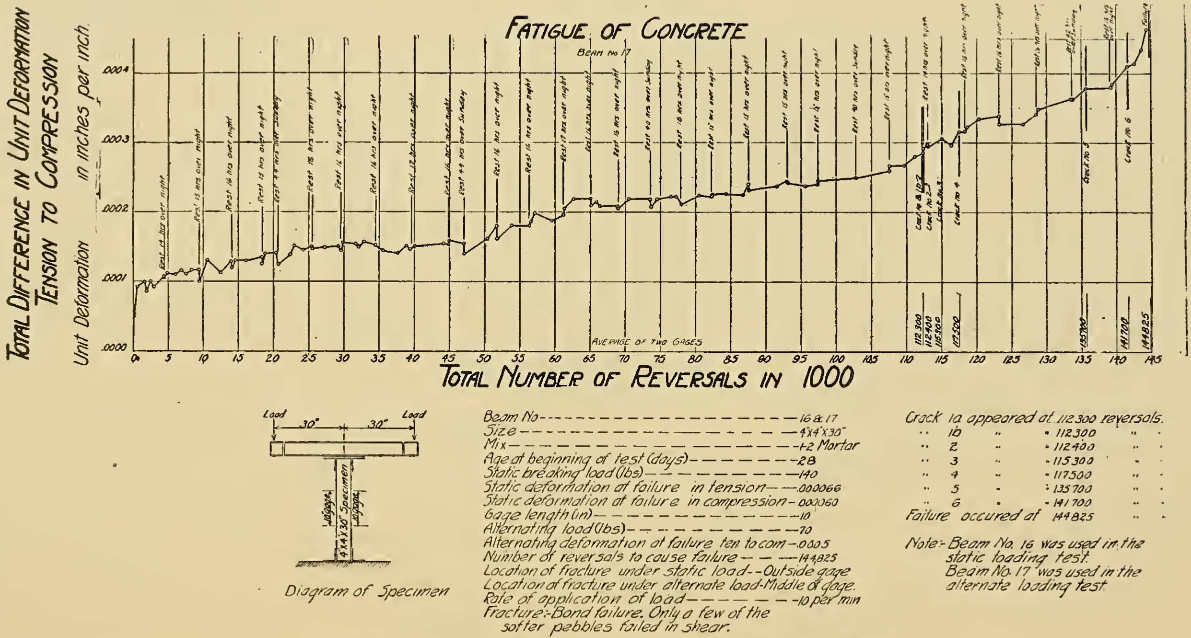
TABLE II.—FATIGUE TEST.

STATIC LOADING.

Beam—No. 28.
Mix—1 : 2.
Age—28 days.
Load applied 30 inches from center line of column.

Load in Pounds.	Unit Deformations, in. per in.	
	Gage No. 1 (Tension).	Gage No. 2 (Compression).
0	.000 000 0	.000 000 0
30	.000 022 0	.000 013 8
40	.000 030 6	.000 013 8
50	.000 033 5	.000 026 4
60	.000 049 4	.000 031 8
70	.000 051 6	.000 032 0
80	.000 070 0	.000 051 0
90	.000 078 6	.000 059 1
100	.000 090 7	.000 065 1
110	.000 097 8	.000 069 6
120	.000 110 5	.000 089 7
130	.000 129 8	.000 103 7
140	.000 136 5	.000 108 9
150	.000 156 0	.000 122 7
160	....	....

NOTE. Beam failed 2½ in. inside of gage after it held a load of 160 lb. for a few seconds.



Beam No—16 & 17
Size—4"X4"X30"
Mix—1-2 Mortar
Age at beginning of test (days)—28
Static breaking load (lbs)—140
Static deformation at failure in tension—.000066
Static deformation at failure in compression—.000060
Gage length (in)—10
Alternating load (lbs)—70
Alternating deformation at failure ten. to com.—.0005
Number of reversals to cause failure—144,825
Location of fracture under static load—Outside gage
Location of fracture under alternate load—Middle of gage.
Rate of application of load—10 per min
Fracture:—Bond failure. Only a few of the softer pebbles failed in shear.

Crack 1a appeared at 112300 reversals.
" 1b " " 112300 "
" 2 " " 112400 "
" 3 " " 115300 "
" 4 " " 117500 "
" 5 " " 135700 "
" 6 " " 141700 "
Failure occured at 144825 "

Note:—Beam No. 16 was used in the static loading test.
Beam No. 17 was used in the alternate loading test.

FIG. 5.—SUMMARY OF FATIGUE TESTS ON SEVERAL BEAMS.

TABLE III.—FATIGUE OF CONCRETE.

SUMMARY SHEET.

4 x 4 x 30 in. Beams .

Mark.	Test.	Mix.	Age at beginning of Test, days.	Gage Length, in.	Static Breaking Load, lb.	Static Breaking Deformations.		No. of Applications of Load per minimum.	Alternate Load.		No. of Reversals to cause Failure.	Alternate Deformation.				Location of Fracture.	
						Tension, in. per in.	Compression, in. per in.		lb.	Per cent of Static.		At Reversals.	Tension to Compression.			Under Static Load.	Under Alternate Load.
													Maximum, in. per in.	Minimum, in. per in.	Average, in. per in.		
1	St.	1 : 2	22	..	160	....	....	..	..	..	......	......	....	....	....	......	....
2	St.	1 : 2	14	..	140	....	....	..	..	..	......	......	....	....	....	......	......
3	St.	1 : 2½	14	..	130	....	....	..	..	..	......	......	....	....	....	......	......
5	St.	1 : 2	7	..	110	....	....	..	.	..	......	......	......	....	....	.....	......
7	St.	1 : 1½	7	..	110	....	....	..	..	..	......	......	....	....	....	......	......
9	St.	1 : 2½	7	..	180	....	....	..	..	..	......	......	....	....	....	......	.....
11	Alt.	1 : 2½	7	..	...	....	....	10	50	28	16 200*	......	....	....	....	.....	......
12	St.	1 : 1½	14	2	220	....	....	..	..	..	......	......	....	....	....	......	......
13	Alt.	1 : 1½	14	2	...	....	....	10	100	45	2 704	2 500	....	....	.000 134	......	In gage
14	St.	1 : 2	14	8	140	....	....	..	..	..	......	......	....	....	....	......	......
15	Alt.	1 : 2	14	10	...	....	....	10	50	36	37 200	37 198	....	....	.000 081 6	......	Out gage
16	St.	1 : 2	28	10	140	(.000 066)	.000 060	..	..	..	......	......	....	....	....	Out gage	......
17	Alt.	1 : 2	28	10	...	....	....	10	70	50	144 825	144 825	....	....	.000 5	......	In gage
18	.	....	..	..	...	....	....	..	..	..	......	......	....	....	....	......	......
19	..	....	.	..	...	....	....	..	..	..	......	......	....	....	....	......	......
20	St.	1 : 1½	7	10	200	.000 190	.000 173 8	..	..	..	......	......	....	....	....	In gage	......
21	Alt.	1 : 1½	7	10	...	....	....	10	100	50	9 280	9 270	.000 241 6	.000 145 3	.000 193 5	......	In gage
22	Alt.	1 : 2	40	10	...	....	....	10	90	64	9 800	5 820	.000 191 5	.000 138 0	.000 164 8	......	Out gage
23	Alt.	1 : 2	40	10	...	....	....	10	105	75	374	......	....	....	....	......	Out gage
24	St.	1 : 2	40	10	140	.000 167 7	.000 144 9	..	..	..	......	......	....	....	....	In gage	......
25	St.	1 : 2	40	10	140	.000 115 7	.000 108 7	..	..	..	......	......	....	....	....	At gage	......
26	St.	1 : 2	28	10	210	.000 195 6	.000 152 9	..	..	..	......	......	....	....	....	In gage	......
27	Alt	1 : 2	28	10	...	....	....	10	105	50	11 900	10 580	.000 273 6	.000 231 8	.000 252 7	......	In gage
28	St.	1 : 2	28	10	160	.000 156 0	.000 122 6	..	..	..	......	......	....	....	....	In gage	......
29	Alt.	1 : 2	28	10	...	....	....	10	80	50	29 600	28 820	.000 194 6	.000 174 4	.000 184 5	......	Out gage
30	Alt.	1 : 2	35	10	...	....	....	10	80	50	26 410	25 610	.000 215 3	.000 133 3	.000 174 3	......	In gage
31	St.	1 : 2	28	10	160	.000 131 0	.000 120 2	..	..	..	......	......	....	....	....	At gage	......
32	St.	1 : 2	28	10	180	.000 217 1	.000 136 4	..	..	..	......	......	....	....	....	At gage	......
33	Alt.	1 : 2	28	10	...	....	....	10	80	47	22 500	22 180	.000 247 2	.000 192 8	.000 220 0	......	In gage
34	Alt.	1 : 2	28	10	...	....	....	10	72	40	3 100	1 900	.000 127 0	.000 082 6	.000 104 8	.. ...	In gage
35	Alt.	1 : 2	28	10	...	....	....	10	90	50	2 520	2 510	.000 321 2	.000 179 5	.000 250 3	......	In gage
36	St.	1 : 2	28	10	180	.000 176 3	.000 136 8	..	..	..	......	......	....	....	...	In gage	..

No. 9 and 11, same mortar.

Test stopped.

to the upper end so that bending stresses are induced so much greater than the direct stresses that the test is practically one of beam action. The photograph shows the apparatus in place and that loads are placed alternately at either end of the bracket by means of walking beams.

After some experience with a rectangular prism the specimen was changed in form to provide enlarged ends for gripping so that fractures were more likely to occur in the gage length. The rapidity of the application of the load was at the rate of 10 cycles per minute. Berry strain gages are applied to both ends of the specimen and observations taken of the deformations from tension to compression and vice versa on the two sides of the specimen.

Fig. 2 shows the sequence of deformations and the set, or failure to return to its original shape, first in tension and then in compression. It is probable that this working of the material through its plastic stage between these sets is the cause of the so-called fatigue.

It has been necessary each night to shut off the electric current which supplies power to the apparatus, and during this period of rest the concrete recovers from the working of the preceding day as shown very nicely in Fig. 3. Table III gives the results of the test up to the present time. The mortar mix ranges from 1 : 1½ and the age of the specimen from 7 to 40 days. After some preliminary experiments the investigations are now being confined to mortar beams of 1 : 2 mix and the age of 28 days.

First of all a twin beam is broken in so-called static loading or when weights are increased on one end of the bracket until the specimen fractures. (See Fig. 4 and Table II.) The fatigue range is then taken as a per cent of the static load, ranging in the test from 36 to 75 per cent of this static load.

Deformations under the normal working of the apparatus were compared to those with the loads at rest to determine the impact or suddenly applied effect of the load. The latter effect is about 1 per cent and may be disregarded. (See Fig. 5.)

GENERAL BEHAVIOR UNDER TEST.

The curves show the phenomena of the test quite clearly. There is an increasing deformation under fatigue until the beam finally ruptures. The number of reversals at rupture varies greatly, and there is apparently no well defined law exhibited. The tendency is toward fewer cycles at rupture almost directly as load is increased. Beams which have a high degree of stiffness seem to have more endurance than those of less stiffness. It is planned for the immediate future that the endurance of beams shall be determined on the basis of the increasing per cent of static load to determine if there is a critical stress similar to that observed by Professor Moore in the case of steel.

The general indications are that ultimate extensibility of the beams may be reached either by a load once applied or by a smaller load repeated a number of times.

PROPORTIONING CONCRETE MIXTURES.

By Duff A. Abrams.*

A discussion preceding a demonstration of actual field proportioning of concrete by Stanton Walker. Mr. Walker has prepared a resumé of the methods used which appears on p. 182 of this volume.

Many different methods have been used and several theories have been advanced as a basis for proportioning concrete. I need only refer to a few of these to refresh your memory. We have the present custom (almost universal, I regret to say) of proportioning concrete by arbitrary quantities; that is, we use, say a 1 : 2 : 4 mix regardless of the particular type of materials available and regardless of the quantity of mixing water used and take little or no account of the strength of concrete produced. Needless to say, this method is extremely crude, and does not recommend itself to thoughtful engineers.

The maximum-density-of-aggregate method has been advocated and used to some extent. Here the endeavor was to grade aggregates in such a way as to give an aggregate mixture of maximum density, and then supply the quantity of cement that was considered necessary to produce the concrete desired for a particular purpose.

Another method which has been used in the proportioning of concrete is based on the voids in the aggregates. The voids in the coarse aggregates were determined, a sufficient quantity of sand was supplied to fill those voids, then a sufficient quantity of cement was used to fill the voids in the combined aggregates. This is a little more rational than the purely arbitrary method. It is not, however, on a scientific basis. It will, in general, give fairly good concrete, but it has been shown by our investigations and the tests of others, that the voids in the aggregate is not the determining factor in concrete mixtures.

Experimental Studies of Concrete.

A great deal of experimental work has been done in endeavoring to arrive at a proper basis for proportioning concrete. Much of the early work of R. Feret,† a French engineer, was devoted to a study of this problem. Feret confined his studies almost entirely to sand mortars, so that it is not easy to apply his methods directly to a concrete mixture.

Allen Hazen** did some very interesting work in his studies of the use of sands in water filtration. He made use of the "effective size" and "uniformity coefficient" of sands. These have undoubtedly served a useful purpose in that connection, but many engineers have endeavored to apply the same factors to concrete aggregates with no obvious reason why they should be applicable. It is apparent that these functions cannot serve any useful purpose so far as concrete aggregate is concerned, since they are based on separations made by one or two sieve sizes and consequently are quite inadequate to show the value for concrete of aggregate which covers a wide range in size. A good idea may be gained of the wide range in sizes in concrete aggregate when we consider that the diameter of a 3-in. pebble is over 500 times that of a particle of sand which passes the 100-mesh sieve.

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*Professor in Charge, Structural Materials Research Laboratory, Lewis Institute, Chicago.

†Ann. Ponts et Chaussees; also Etude Experimentale du Ciment Arme, 1906.

**24th Annual Report Mass. State Board of Health; reprinted in "State Sanitation"; Harvard Univ. Press, v. 2, p. 232, 1917.

The work of Fuller and Thompson* is a source of valuable information. However, our experiments have shown that the grading given by Mr. Fuller in his "maximum-density curve" is too coarse for the ordinary mixtures; in other words, for the usual quantities of cement, the grading he suggests gives a harsh-working concrete. Another corollary of Mr. Fuller's method indicates that the aggregates should be separated into a number of sizes and recombined in order to approximate this so-called maximum-density curve. Our investigations show that the separation of aggregates into a number of different sizes and recombining, is seldom, if ever, necessary or desirable; and especially in the case of the fine materials involves a great expense and many difficulties. I believe that this method has been used to some extent, but the practical difficulties in the way have barred it from general application.

A number of other experimentors have been studying this problem during the past few years. I might mention Captain L. N. Edwards(¹), R. B. Young(²), Prof. A. N. Talbot(³), F. L. Roman(⁴), formerly of the Illinois Division of Highways, and R. W. Crum(⁵), of the Iowa State Highway Department. All these men have furnished valuable information, so that we are able to approach the subject from many different angles.

At the Structural Materials Research Laboratory we have been engaged in experimental studies of concrete and concrete materials for nearly 8 years. These tests are being carried out through the co-operation of the Portland Cement Association and Lewis Institute. It was apparent at the beginning of this work that a proper basis for proportioning concrete was not available; consequently, among our first investigations were a number of studies of the general subject of proportioning. These studies have been continued up to the present.

The studies carried out thus far which deal with the many phases of proportioning concrete reach a total of about 100,000 tests. These tests fall under the following categories:

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*Trans. Am. Soc. C. E., v. 59, 1907; also "Concrete Plain and Reinforced," by Taylor and Thompson, 1916.

[1] Proc. A. S. T. M., v. XVII, Part II, p. 301, 1917; also Proc. v. XVIII, Part II, p. 303, 1918.

[2] Proc. A. S. T. M., v. XX, Part II, p. 137, 1920.

[3] Proc. A. S. T. M., v. XXI, Part II, p. 940.

[4] Eng. & Cont., May 5, 1915, p. 403; also Bull. 14, Ill. State Highway Department, 1917.

[5] Proc. A. S. T. M., v. XIX, p. 458, 1919.

1. Standardization of test methods,
2. Effect of grading of aggregates,
3. Effect of consistency of concrete,
4. Effect of quantity of cement,
5. Relative merits of different types of fine and coarse aggregates,
6. Comparison of concrete and mortar tests,
7, Effect of curing condition,
8. Effect of age of concrete,
9. Effect of powdered admixtures in concrete,
10. Studies of methods of measuring plasticity or workability of concrete,
11. Comparisons of compressive strength, tension, flexure, wear, yield and absorption of concrete.

THEORY OF CONCRETE MIXTURES.

The real problem in proportioning concrete mixtures is to take advantage of the characteristics of the materials available. At the present time aggregate materials cannot be transported long distances; we may know ever so much about the ideal characteristics of sand and coarse aggregates, but we may not be able to get that type of aggregate. In other words, we must make the best we can of the materials available.

Our investigations have led us to the following fundamental principles:

1. The strength of a concrete mixture depends on the quantity of mixing water in the batch, expressed as a ratio to the volume of cement *so long as the concrete is workable*, and the *aggregates are clean and structurally sound.*

2. The effect of differences in the quantity of cement is reflected by differences in the water-ratio.

3. The sieve analysis of the aggregate is the basis on which proportioning must be done. There is an intimate relation between the size and grading of the aggregate and the quantity of water required to produce concrete of a given workability.

4. It is not necessary, or desirable, that the aggregate be proportioned according to any fixed grading; wide variations in gradings of aggregate may occur without affecting the quantity of mixing water or the quality of the concrete.

5. Plasticity or workability is an essential requirement of concrete for structural purposes.

The sieve analysis expresses the life history of the material, so far as its concrete-making properties are concerned. It is only necessary for us to interpret this information in order to use the material to the best advantage.

The mixing water in concrete serves two distinct purposes, although we do not always recognize those distinctions. One purpose is to supply the water necessary for the hydration of cement. The second purpose is to produce a plastic mixture. I do not believe anyone has determined just how much water is required for hydration, but such information as we have indicates that it is a relatively small percentage as compared to the quantity we must use in the concrete; probably not more than a quarter or a third of the water that is usually placed in the concrete. The excess of water over and above that required for hydration of the cement simply goes to produce a plastic mixture and is removed later by evaporation.

The water-ratio seems to be the factor which ultimately governs the strength and wearing resistance of the concrete. If we use one cubic foot of water (about 7½ gal.) to each sack of cement, we would have a water-ratio of 1; a smaller quantity of water per sack would give a water-ratio of less than 1, etc. The usual water-ratio for ordinary conditions is about 0.8 or 0.9, or about 6 to 6¾ gal. per sack of cement. In dealing with water-ratio we disregard water absorbed by the aggregates, although water mechanically held by the aggregate would be considered. A great many tests carried out in our laboratory, and those made by others, have shown that the water-ratio is a reliable factor in studying the characteristics of concrete. Anything that can be done in improving the grading of the aggregate, in the use of more cement or different methods of finishing or placing concrete, in order that it may be placed with a lower quantity of mixing water—all of these things are simply means to an end; namely, to produce a plastic concrete with a lower water-ratio.

This brings us to a point which is generally overlooked in our building codes and other documents of that kind. It seems to be the opinion that increasing the quantity of cement is the cure for all the difficulties of weak or inferior concrete. However, if adding cement is not at the same time accompanied by a reduction in the water-ratio, it does not accomplish any useful purpose. The water-ratio may be changed due to changes in the relations of the quantity of cement, grading of aggregate, or changes in relative consistency of the concrete; however, we arrive at the same result, indicating that the water-ratio is the thing that actually controls the strength and other properties of concrete.

It is not necessary, or desirable, that the aggregate be proportioned according to any fixed grading; wide variations in gradings of aggregate may occur without affecting the quantity of mixing water or the quality of the concrete.

PLASTICITY OF CONCRETE.

The question of plasticity or workability of concrete is a very important one. We are hedged about on all sides by the requirement of plasticity. We could not use concrete at all if it were not for the fact that it can be mixed in a plastic condition and placed as such. We must put in enough water to make our concrete plastic so that it can be worked into place. Now, as to the method of measuring or controlling the plasticity of concrete; this is one of the questions which is giving engineers more concern today than probably any other one thing. A number of methods have been suggested and a great deal of experimental work is now under way. The method which probably has met with most common use, because of its simplicity, is the slump test. This test will be described more in detail in Mr. Walker's discussion. The Bureau of Standards is responsible, I believe, for a method which is known as the flow-test, in which a sample of the concrete is placed on a horizontal table which is raised and dropped 15 times in about 10 seconds, thus flattening out the concrete. The spread or increase in diameter is taken as a measure of the plasticity of the concrete.

There is a great need for constructive research in developing a satisfactory and practical method of determining the plasticity of concrete. Any method which is used for this purpose in the field should not be interpreted too strictly. In other words, if our specification indicates that the concrete should be mixed to a slump, say, of 6 in., we would not be justified in rejecting a batch of concrete simply because it gave a slump of 7 in. The purpose is to take care that the concrete does not habitually go beyond the quantity fixed by the specification. In most building construction, if you attempted to make a slump test, you would get nearer 12 in. than anything else. That is the type of concrete that we must guard against, and I believe that the big thing this test can do is to bring home the importance of the water control on building construction. I know that a great many people are skeptical as to the practicability of such control; some may still doubt whether an excess of water does produce an inferior concrete; but I can say with all the emphasis of which I am capable that I am very sure it does, and that in many cases the concrete produced on the job has a strength of probably not more than 20 to 30% of the strength it should have, and probably not more than 60 or 70% of the strength upon which the design was based. So, under these conditions we should not be surprised to find inferior concrete, and we do find it too often.

APPLICATION OF PRINCIPLES.

The foregoing principles lead us to a number of practical features of concrete-making.

The quantity of mixing water used to produce concrete of the condition of workability necessary is of major importance. Every care should be taken to restrict the mixing water to a minimum. An excess of mixing water produces the same result as omitting a large portion of the cement from the batch.

Methods have been developed whereby aggregates may be scientifically proportioned on the basis of sieve analysis without separation into large number of sizes. Aggregates may be used with good results regardless of the fact that they do not conform to our older ideas of what concrete aggregate should be. For example, the Iowa State Highway Department has embodied in their specifications for concrete roads a method whereby so-called pit run gravels, can be used in almost any proportions in which they occur, so long as they are clean, regardless of the fact that there may be a great excess of sand. Their method requires an increase in the quantity of cement in proportion to the quantity of sand in excess of a certain minimum. The specifications would permit the use of a mixture of one part sand and one part coarse aggregate or of a sand only, providing the cement content were increased to such an extent as to produce concrete of the strength and wearing resistance aimed at by the specification. This instance is not an isolated one. The Illinois Division of Highways permits the same thing. In many sections of Kansas and Nebraska great quantities of sand and very little coarse aggregate are available; here a 1: 3 sand mixture has been used with good results. Of course this procedure would have been frowned upon a short time ago, but I believe it has proven its value and will become general practice in the future.

Our problem is to put together the aggregates available in order to get the best concrete mixture we can for a given cost or at a minimum cost.

QUALITY OF CONCRETE.

A great deal has been said with reference to the compressive strength of concrete and we have come to accept the compressive strength as a measure of the other desirable qualities of concrete.

The strength of concrete is an important factor in building construction and in other types of construction; however, the quality of concrete has a wider bearing than strength. A structure which is exposed to the weather, with wide ranges of temperature, wide variations in moisture content, and probably exposed to other destructive agencies, must have a very considerable degree of resistance if it is to give a good account of itself. That is entirely aside from the matter of strength. It is true, however, that the strength of concrete reflects to a very large degree the ability of concrete to withstand these other agencies. Our experiments have shown that so far as wearing* resistance is concerned, it is true that the strength is the proper measure of the quality. The tests which I showed here yesterday indicated that the flexural strength** of concrete is not influenced in exactly the same way as the compressive strength, but the relation is in the same direction. The variation in water content did not affect the modulus of rupture so much as the compressive strength. However, we may consider that the compressive strength of concrete is an entirely satisfactory measure of its quality.

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*See "Effect of Curing Condition on the Wear and Strength of Concrete," Bull. 2, Structural Materials Research Laboratory; also "Wear Tests of Concrete," Proc. Am. Soc. Testing Mat., 1921.

**Flexural Strength of Plain Concrete; see page 20 of this volume.

TABLES OF PROPORTIONS AND QUANTITIES.

As a result of our experimental studies of concrete and concrete aggregates we have prepared a set of tables of quantities and proportions of materials for concrete of a given strength.* Some of these tables appeared in the Joint Committee report; others have been added. The purpose of these tables is to show proportions and quantities which, under average conditions, may be expected to give concrete of a given compressive strength under normal conditions of curing.

Their chief value is in the selection of proportions for preliminary investigations and in making changes in proportions required by variations in materials, after a basic mixture has been determined upon. It is not expected that the tables will give as exact information as may be obtained by preliminary investigations of materials and mixtures, followed by control tests made during the progress of the work.

STANDARD TESTS OF CONCRETE AND CONCRETE MATERIALS.

I wish to add a few words on the work of standardization which is being done by the American Society for Testing Materials. During the past year or two that Society has published in their "Standards" and "Proceedings" the following *new* standards or tentative standards which are of interest to all concrete engineers:

1. Standard Method of Making and Storing Specimens of Concrete in the Field;
2. Tentative Method of Making Compression Tests of Concrete;
3. Tentative Specifications for Workability of Concrete for Concrete Pavements;
4. Tentative Method of Test for Sieve Analysis of Aggregate for Concrete;
5. Tentative Method of Test for Organic Impurities in Sands for Concrete;
6. Tentative Method of Securing Specimens of Hardened Concrete from the Structure;
7. Standard Method of Test for Unit Weight of Aggregate for Concrete;
8. Tentative Specifications for Concrete Aggregate;
9. Tentative Method of Test for Voids in Fine Aggregate for Concrete.

Those of you who have not looked up these standards recently will be well repaid for doing so. Many of the concrete tests made in the past are of little or no value, due to the fact that sound principles of testing were violated. It is frequently impossible to properly interpret indications of concrete tests, due to lack of proper standard methods of manipulation. The tentative standard method for making tests of concrete should be consulted and followed so far as possible by all who have occasion to make tests. In those standards the details are given whereby consistent results can be secured in different laboratories and in different sections of the country; something we have not been able to do heretofore.

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*"Quantities of Materials for Concrete," by Duff A. Abrams and Stanton Walker, Bull. 9, Structural Materials Research Laboratory.

We have been floundering around for 40 years and for the first time we now have some semblance of a standard method for making sieve analysis of aggregates. Heretofore practically every laboratory used a different set of sieves or used a different method of recording sieve analysis, and consequently the results have been so divergent that it has been almost impossible for one person to make anything out of the work done by others; standard sieves and methods are very important steps in advance.

CONCLUDING REMARKS.

Considerable progress is being made in many directions in improving and standardizing concrete tests and practices, all of which will exert an important influence on the quality of work, and will in the near future result in a higher batting average of good concrete.

FIELD TESTS TO DETERMINE PROPORTIONS FOR CONCRETE*.

BY STANTON WALKER**.

Considerable interest has been shown in the methods for the design of concrete mixtures developed at the Structural Materials Research Laboratory through the co-operation of the Portland Cement Association and Lewis Institute. These notes were prepared with the view of bringing about a better understanding of the applications of these methods to the control of the quality of concrete in the field. Professor Abrams has just told you something of the underlying principles of the theory of concrete mixtures. I will outline briefly a procedure for applying this theory to proportioning concrete. These methods may be easily applied in the field by the average contractor or engineer, without his delving into the so-called "mysteries" of the theory of concrete proportions.

My discussion of these methods will involve the description of a diagram from which proportions of materials for concrete of a given strength may be determined; a description of the tests necessary for the application of these methods and an illustration of the necessary calculations for determining the proportions of materials for a given strength.

PURPOSE OF METHODS.

The chief purposes which these methods serve may be briefly described as follows:

1. To furnish a guide in the selection of mixtures to be used in preliminary investigations of the strength of concrete.
2. To indicate proportions which may be expected to produce concrete of a given strength under average conditions where control tests are not made.
3. To furnish a basis for comparing the relative economies of concretes made by combining aggregates of different sizes, when the workability and strength of the resulting concrete are taken into account. (In other words, it is not necessary to restrict aggregate sizes to those usually mentioned in specifications.)
4. To furnish a basis for making changes in proportions to compensate for variations in size and grading of aggregates and workability of concrete during the progress of the work.

It should be emphasized that the chief value of the methods described here and in various publications of our Laboratory is in the selection of proportions for preliminary investigations and in making changes in proportions required by variations in materials, after a basic mixture has been established. It is not expected that they will give as exact information as may be obtained by preliminary investigations of materials and mixtures, followed by control tests made during the progress of the work.

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*Notes prepared from a discussion coupled with a practical demonstration of methods before the 1922 Convention.

**Associate Engineer, Structural Materials Research Laboratory, Lewis Institute, Chicago.

BASIS OF METHODS.

As Professor Abrams pointed out, the results of investigations carried out at this Laboratory and elsewhere, show that the strength of concrete for given materials, conditions of curing, methods of testing, etc., is a function of the quantity of water in the batch expressed as a ratio to the volume of cement *so long as the concrete is plastic and workable.* This has become known as the water-ratio theory of proportioning concrete. The water-ratio, and hence the strength of concrete involves the following factors:

1. Size and grading of aggregate,
2. Quantity of cement (the mix),
3. Plastic condition of the concrete.

Figure 1 was constructed to show the average interrelation of compressive strength of 6 by 12-in. cylinders stored in a moist place and tested at the age of 28 days, size and grading of aggregate, quantity of cement, and plasticity of concrete, as indicated by the slump test. This diagram is based on the results of many investigations carried out at the Structural Materials Research Laboratory during the past 8 years. The 6 x 12-in. concrete cylinders were made and tested under standardized methods similar to those described in "Tentative Methods of Making Compression Tests of Concrete," of the American Society for Testing Materials.

LIMITATIONS OF METHODS.

These methods furnish a guide in the selection of concrete mixtures and for comparing different mixtures within the following limitations:

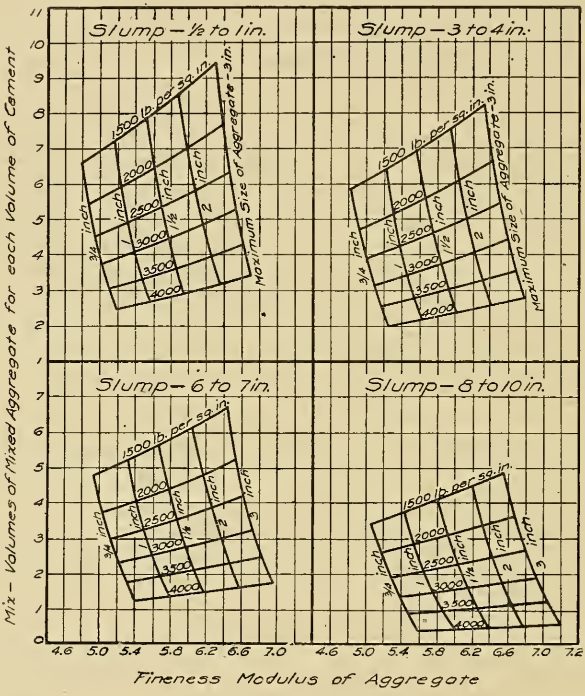
1. Concrete must be plastic or workable,
2. Aggregates must be clean and structurally sound,
3. Cement must meet requirements of the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials,
4. Allowance must be made for temperatures below normal; low temperatures during the first few days of curing greatly reduce the strength of concrete,
5. If the proportions to be used in the work are selected without preliminary tests of the materials and control tests are not made during the progress of the work, the mixtures for a 6 to 7-in. slump should be used.

INFORMATION NECESSARY FOR USE OF DIAGRAMS.

The following information is needed in order to determine the proportions for concrete of a desired strength:

1. Sieve analyses of aggregate made in accordance with standard methods,
2. Fineness modulus of aggregates (calculated from sieve analysis),
3. Maximum size of aggregate,
4. Unit weight of fine, coarse, and mixed aggregates,
5. Plastic condition of concrete required for work, as represented by the slump test,
6. Desired compressive strength of concrete at 28 days,
7. Knowledge of cleanness and structural soundness of aggregate.

Fig. 1
Relation of Size and Grading of Aggregate
And Quantity of Cement to Strength of Concrete.



METHODS OF TEST.

The tests from which the above information can be determined can easily be made in the field. Only a brief description of them need be given here. More complete details may be found in the "Proceedings" and "Standards" of the American Society for Testing Materials.

Sampling of Aggregates. It is of utmost importance that representative samples of the aggregates be selected for test. Samples for sieve analyses should be selected by quartering or by use of a sampler. For the sieve tests on fine aggregate a 500-gram sample should be used. The weight in grams of the sample for coarse aggregates should be about 3000 times the maximum size of particle measured in inches.

Sieve Analysis. The sieve analysis should be made in accordance with the "Tentative Method of Test for Sieve Analysis of Aggregate for Concrete" of the American Society for Testing Materials. The test is made by determining the cumulative percentages finer or coarser than each sieve.

A convenient set of sieves for field use, consisting of No. 50, 16, 4 and ¾-in. sieves, can be made to nest in a cylindrical container about 4½ in. in diameter and 12 in. long. The percentages for the omitted sieves may be obtained by interpolation (arithmetically or graphically—preferably graphically), with sufficient accuracy.

TABLE I.—SIEVES FOR AGGREGATE TESTS.

Square-mesh woven wire sieves.

Sieve Number or Size.	Size of Square Opening, inches.
100	0.0059
50*	0.0117
30*	0.0232
16*	0.0469
8	0.0937
4	0.187
⅜-in	0.375
¾-in	0.75
1-in.*	1.00
1½-in	1.50
2-in.*	2.00
3-in	3.00

* Sieves No. 50, 30 and 16 are identical with No. 48, 28 and 14, used in earlier reports of the Structural Materials Research Laboratory. The 1- and 2-in. sieves are half-size and are usually omitted from our reports.

Fineness Modulus. The fineness modulus of an aggregate is the summation of the percentages in the sieve analysis divided by 100, omitting the 1-in. and 2-in. sieves, when the sieve analysis is expressed as cumulative percents coarser than each sieve. This function is a measure of the grading of the aggregate. Its use in the calculations for proportions will be illustrated.

Maximum Size. The size of aggregate to be considered as the upper limit is determined by applying the following rules:

1. Not less than 15% shall be retained between the sieve which is considered as the maximum size and the next smaller sieve.
2. Only the sizes given in Table 1 shall be considered.

Unit Weight of Aggregate[*]. The weight of aggregate per unit volume is determined by puddling the aggregate into a cylindrical measure having height equal to diameter, with a ⅝-in. round rod pointed at the lower end. For sand, a measure of 1-10 cu. ft. capacity is recommended. For coarse aggregate and mixed aggregate a measure of ½ cu. ft. is recommended.

Slump Test. The slump is used to denote the plasticity or workability of the concrete. The method of making this test is described in the "Tentative Specifications for Workability of Concrete for Concrete Pavements," of the American Society for Testing Materials, as follows:

"In determining workability, the newly mixed concrete shall be placed in a truncated *cone-shaped* metal mold 12 in. high, 8 in. in diameter at the base, and 4 in. in diameter at the top, and provided with handles at the sides. The concrete shall be lightly tamped with a rod as it is placed in the mold which, when filled, shall be immediately removed and the slump or settlement of the concrete noted."

The slump is the settlement in inches of the truncated cone of concrete from its original height of 12 in.

Cleanness of Aggregate. It is assumed that the aggregates will be clean; that is, free from injurious amounts of shale, alkali, organic matter and other deleterious substances. The test for organic impurities in sands may be made by digesting a sample in a 3% solution of sodium hydroxide[*] and observing the color of the supernatant liquid. A dark color indicates the presence of such quantities of organic impurities that the sand should be rejected, or subjected to rigid tests before use.

CALCULATION OF PROPORTIONS.

For an illustration of the method of calculating the proportions with the aid of Fig. 1, consider the following problem:[**]

With fine and coarse aggregates of the characteristics shown in Tables II and III, let us determine the proportions for concrete having a slump of 6 to 7 in. and a compressive strength at 28 days of 2000 lb. per sq. in.

The application of the rule given above to the sieve analysis of the pebbles show them to be graded up to 1½ in., since the percentage coarser than the next smaller size (1 in.) is greater than 15%. Therefore, the values for mix and maximum permissible fineness modulus are read on the diagram for 6 to 7 in. slump, from ordinate and abscissa corresponding to the point fixed by the intersection of the contour for 2000-lb. concrete with that for aggregate having a maximum size of 1½ in. For these conditions a mix of 1 : 4.4 and a fineness modulus of 5.75 are found.

It is now required to determine the proper proportions of fine and coarse aggregate, and for convenience to translate the mixture expressed in terms of volumes of *mixed* aggregate (the *real* mix) to one expressed in terms of separated aggregates (the *nominal* mix).

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[*] See "Standard Method of Test for Unit Weight of Aggregate for Concrete," A. S. T. M. Standards, 1921, p. 639.

[*] See Proc. Am. Soc. Testing Mat., 1921, p. 585; also "Concrete Highway Magazine," Feb., 1918.

[**] For other examples of methods of calculating proportions, see "Applications of Abrams' Water-Ratio Theory to Proportioning Concrete," Proc. A. C. I., 1920.

Since we know the fineness moduli of the fine, coarse, and mixed aggregate, the ratio of fine to coarse may be found by substitution in the following simple formula:

$$r = \frac{m_c - m}{m_c - m_f}$$

TABLE II.—SIEVE ANALYSES OF AGGREGATE.

The sieves were of square mesh wire cloth manufactured by the W. S. Tyler Company. These sieves are recommended by the Bureau of Standards and used in the "Tentative Method of Test for Sieve Analysis of Aggregate for Concrete" of the American Society for Testing Materials.

Sieve No.	Size of Square Opening, inches.	Amounts Coarser than Each Sieve, per cent by weight.	
		Sand.	Pebbles.
100	0.0059	99	100
50*	0.0117	90	100
30*	0.0232	61	100
16*	0.0469	32	100
8	0.0937	18	98
4	0.187	5	95
⅜	0.0375	0	70
¾	0.75	..	35
1	1.00	..	17
1½	1.5	..	0
	Fineness modulus‡	3.05	6.98

* The clear opening of sieves No. 50, 30 and 16 are identical with No. 48, 28 and 14 used in earlier reports of the Structural Materials Research Laboratory.

† Omitted in calculation of fineness modulus.

‡ Sum of per cents in sieve analysis divided by 100.

TABLE III.—MISCELLANEOUS DATA OF AGGREGATE.

Item.	Sand.	Pebbles.
Unit weight, lb. per cu. ft.	112	108
Organic impurities, by colorimetric test	trace	trace
Silt, per cent by weight	1.5	...
Aggregates appear to be made up of hard particles suitable for use in concrete.		

where

r = ratio of fine aggregate to volume of fine and coarse measured separately,
m = fineness modulus of mixed aggregate,
m_c = fineness modulus of coarse aggregate,
m_f = fineness modulus of fine aggregate.

Substituting the conditions for this problem, we have

$r = \frac{6.98 - 5.75}{6.98 - 3.05} = \frac{1.23}{3.93} = 0.31$, the ratio of volume of fine aggregate to volume of fine and coarse measured separately.

In order to translate the *real* mix to the *nominal* mix, it is necessary to determine the unit weight of the aggregate mixed in the proportions of 31% sand and 69% pebbles. Assume that this is found to be 127 lb. per cu. ft. To determine the relation which the volume of mixed aggregate bears to the volume of sand and pebbles measured separately, calculate the volume after mixing, of 1 cu. ft. of fine and coarse aggregate measured separately, as follows:

0.31 cu. ft. sand $= 0.31 \times 112 =$ 34.8 lb.
0.69 cu. ft. pebbles $= 0.69 \times 108 =$ 74.8 lb.
1.00 cu. ft. aggregate measured separately = 109.6 lb.
or 0.86 cu. ft. of mixed aggregate.

Dividing 4.4 by the ratio 0.86, we find the volume of separated materials for one volume of cement to be 5.1.

The volume of sand $= 0.31 \times 5.1 = 1.58$

The volume of coarse aggregate $= 0.69 \times 5.1 = 3.52$

The nominal mix is therefore 1 : 1.58 : 3.52.

Comparison of this result with the one given in the tables of "Proportions for Concrete of Given Compressive Strength at 28 days" published in the 1921 Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete shows a very close agreement—the table giving a mix of 1 : 1.7 : 3.4 for these conditions. This slight discrepancy is explained by the fact that the fineness moduli of the aggregates assumed for this problem are somewhat different than those used in the tables as representing average conditions.

The proper quantity of mixing water may be found by making the slump test described above. For these conditions, the water-ratio would be about 0.90; or about 6¾ gal. of water per sack of cement.

The quantity of materials per unit volume of concrete can be estimated from the weight and volume of materials in the cone. The volume of a 4 x 8 x 12-in. truncated cone is 0.217 cu. ft. Say that the material in the cone is found to weigh 32.0 lb. The weight per cubic foot of the concrete would therefore be 148 lb.

The weight of concrete produced by 1 cu. ft. of mixed aggregate is then determined as follows:

1 cu. ft. mixed aggregate = 127 lb.
$\frac{1}{4.4}$ cu. ft. cement = 0.227 cu. ft. = 21.3 lb.
(Assuming cement to weigh 94 lb. per cu. ft.)
0.90 x 0.227 cu. ft. water = 0.205 cu. ft. = 12.8 lb.
Total = 160.1 lb.

One cu. ft. of mixed aggregate therefore produces 1.08 cu. ft. of concrete. This value (1.08) is referred to in the various publications of this Laboratory as the "Yield." It may be used readily to calculate the quantities of materials per cubic yard of concrete as follows:

27 cu. ft. (1 cu. yd.) divided by 1.08 gives 25 cu. ft. as the volume of mixed aggregate per cubic yard of concrete; and 25 divided by 4.4 gives 5.7 as the cubic foot of cement per cubic yard of concrete.

25 cu. ft. of mixed aggregate, produces 29 cu. ft. of sand and pebbles measured separately (see above).

The volumes of separated materials are as follows:

$$0.31 \times 29 = 9.0 \text{ cu. ft. sand}$$
$$0.69 \times 29 = 20.0 \text{ cu. ft. pebbles.}$$

PRECAUTIONS TO BE OBSERVED IN THE FIELD.

The methods of applying the principles outlined above in the field will vary somewhat, of course, with the conditions on the job. It is felt, however, that a few general precautions to observe may be pointed out which will be of value.

The calculations for the proportions are based on volumes of dry aggregates measured by puddling in a measure. Therefore, the relation between the volumes measured in this way and the method employed for the large batch in the field must be established. A simple way to do this is to calibrate the measuring device by determining the average weight of its contents and calculating the volume of materials contained in it, with proper allowance for moisture content. For example: the average wheelbarrow load of sand may be found to weigh 275 lb. Say the sand contains 5% of moisture. Then the net weight of the sand is 262 lb. For material having a unit weight of 112 lb. per cu. ft. the volume of sand in this case is 2.3 cu. ft. This calibration should be checked up from time to time as the conditions of the aggregate change. For larger measuring devices, where it is not practical to weigh the entire contents, a modification of this method may be used with sufficient accuracy. The *corrected* cubical contents of the box, hopper or other measuring device may be determined by multiplying the volume of the container by the ratio of the unit weight of the materials measured loose and in the condition in which they are used, to the unit weight determined on dry materials by puddling in a suitable measure. The moisture content must be subtracted from the apparent weight of the aggregate.

The quantity of water for the batch should be determined by making the slump test on concrete from the mixer batches. In this way the allowance for moisture content is automatically taken into account. The quantity of mixing water should be changed from time to time as the moisture content of the aggregate changes.

CONCLUDING REMARKS.

I believe that the economy which may be effected by the use of intelligent methods of proportioning far overbalance the expense involved in applying such methods.

In closing I wish to acknowledge our indebtedness to The W. S. Tyler Company, of Cleveland, for furnishing sieves and other apparatus, and to Mr. W. B. Newberry, of the Sandusky Portland Cement Company, for arranging the concrete materials, scales, balances, etc., used in the demonstration.

Committee Reports Presented to the
Eighteenth Annual Convention
American Concrete Institute

REGULATIONS GOVERNING THE FORM BUT NOT THE SUBSTANCE OF STANDARDS.*

Application 1. *APPLICATION*—These regulations shall be retroactive with respect to all existing standards. Standing Committees shall make an earnest effort to comply with them. Departures therefrom shall not be made by Standing Committees except on what they believe to be strong grounds, and then only after consultation with the Committee on Form of Standards. In case of a disagreement with the Committee on the form of standards, the matter shall be referred to the Advisory Committee, whose decision shall be accepted.

2. *ARRANGEMENT OF MATERIAL.*—The arrangement of material and the designation of sub-titles, etc., shall conform in general to the following:

I. GENERAL. (Sub-title.)
    (*A*)
    (*B*) } (Principal sections under sub-title.)
    (*C*)
    (*a*), (*b*), (*c*), (*d*), etc. (Sub-divisions of a section. Lettering to run consecutively throughout one section only.)

II. MATERIALS.
    (*A*)
    (*B*)
    etc.

III. DESIGN.

IV. DIMENSIONS, WEIGHTS, ETC.

V. CONSTRUCTION AND MANUFACTURE.

VI. WORKMANSHIP.

VII. SPECIAL REQUIREMENTS.

VIII. PACKING, MARKING AND SHIPPING.

IX. INSPECTION AND TESTS.

Sub-titles that do not apply to any given standard may be omitted.

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* These regulations, drawn up by American Concrete Institute Committee G-3 on Form of Standards, conform in general to those already adopted by the American Society for Testing Materials. They have been approved by the Board of Direction of the Institute as a means of attaining uniformity in Institute Standards in respect to form only and are thus issued for the guidance of Institute Committees.

Numbering Paragraphs.

3. *NUMBERING PARAGRAPHS.*—Individual paragraphs in each standard shall be numbered consecutively, irrespective of sub-divisions under which they appear.

Marginal Headings.

4. *MARGINAL HEADINGS.*—Some descriptive word or heading should be placed in the margin opposite each numbered paragraph, briefly indicative of the subject matter.

Units of Measurement.

5. *UNITS OF MEASUREMENT.*—Weights and measures shall be expressed in the English System. Temperature shall be expressed in degrees Fahrenheit.

Abbreviations.

6. *ABBREVIATIONS*—When abbreviations are used they shall conform to the following:

The policy should be to abbreviate too little rather than too much. Terms which seldom occur should in general not be abbreviated.

(a) *Units of Length.*

Foot	ft.
Inch	in.
Linear	lin.
Meter	spell out
Mile	spell out
Yard	yd.

(b) *Units of Area.*

Square	sq.
Square foot	sq. ft.
Square inch	sq. in.

(c) *Units of Volume.*

Barrel	bbl.
Bushel	bu.
Cubic	cu.
Gallon	gal.

(d) *Units of Weight.*

Ounce	oz.
Pound	lb.
Ton	spell out

(e) *Units of Time.*

Afternoon	p. m.
Day	spell out
Forenoon	a. m.
Hour	hr.
Minute	min.
Month	spell out

(e) *Units of Time.*

Second	sec.
Week	spell out
Year	spell out

(f) *Units of heat.*

Degree	°
Fahrenheit	F.

(g) *Miscellaneous Technical Terms.*

Birmingham wire gage	B. w. g.
Browne & Sharpe (gage)	B. & S.
Chemically pure	c. p.
Degree (angular measure)	deg.
Diameter	spell out
Revolutions per minute	r. p. m.
Specific gravity	sp. gr.
Tensile strength	tens. str.
United States (gage)	U. S.

(h) *Miscellaneous General Terms.*

Figure	Fig.
Number	No.
Per	spell out
Per centum	per cent
Proceedings	Pro.
Plate	spell out
Table	spell out
Transactions	Trans.
Volume	Vol.

Abbreviations.

(*i*) Use abbreviations only after nouns denoting a definite quantity, except in tabular work. For example: "The tensile strength is 45,000 *lb. per sq. in.*"; but "The tensile strength in *pounds per square inch is* ......"

(*j*) When terms are used in an abstract or descriptive sense, they shall not be abbreviated. "The work should be done in the *forenoon*", not "in the *a.m.*"

(*k*) Use a period after each abbreviation, except after *per cent*, and as noted in sub-division (*m*).

(*l*) All abbreviations shall be used in the singular. Thus, "two inches" shall be abbreviated "2 in."; not "2 ins."

(*m*) *Compound Words.*—The abbreviations for compound words, when used, shall be formed by connecting the abbreviations of the separate words of a hyphen, and omitting the period preceding the hyphen. Thus, "ft-lb., in-lb."

(*n*) *Symbols.*—Avoid the use of symbols. Do not use (′) or (″) in either text or tables; their use is permissible in illustrations. The symbol (%) shall not be used in the text, but may be used in tables when lack of space requires it.

(*o*) The word "percentage" shall be used when not following a number. Thus, "the *percentage* of reinforcement shall be"; not "the *per cent* of reinforcement shall be." But, "0.77 *per cent* of reinforcement."

(*p*) In expressions like the following, omit the degree mark after the first figure: "75 to 80° F." In a table heading, use "Temperature, *deg. Fahr.*"

(*q*) In expressing dimensions, use the following form: "2 by 4 in. in section;" not "2 x 4 in. in section," nor "2 in. by 4 in. in section."

(*r*) Spell out the names of the months: as, "January 25." Do not use the form "January 25th."

(*s*) In text, do not abbreviate "namely" and "that is".

(*t*) Spell out names of companies, railroads, etc., using the ampersand (&) only between proper names. Abbreviate "Company" in firm names. For example: "Brown & Sharpe Manufacturing Co.," "Philadelphia & Reading Railway Co."; but, "American Steel *and* Wire Co."

Numerals.

7. *NUMERALS.*

(*a*) Roman numerals should be used in designating tables and plates: thus, "Table VI"; not "Table 6." Arabic numerals should be used in designating figures: thus, "Fig. 3"; not "Fig. III."

Numerals.

(*b*) Spell out all numbers from one to twelve, with the following exceptions:

1. Use numerals when the quantity is partly or wholly fractional: as, 1.15, $1\frac{1}{2}$, $\frac{1}{3}$.

2. Use numerals when followed by an expression having a standard abbreviation: as, 1 in., 6 lb., etc.; except where the statement is vague in nature, in which case neither numerals nor abbreviations shall be used: as "about *six pounds*, etc.

3. If for any reason the standard abbreviation of the expression following the number is not used, or, if the expression does not admit of abbreviation (as mile, ton, etc.) the use of numerals shall be optional, unless covered in the following paragraphs.

4. In contrasted statements, if some numbers must be numerals, use numerals for all, as "2 miles and 16 miles."

5. In a series of connected numerical statements implying precision, use numerals: as, "2 years, 5 months, 3 days." The use of numerals (especially the "1") is not recommended for numbers occurring in precise statements similar to the following: "By connecting the *two* columns", "shall consist of *two* bars."

6. Use numerals after abbreviations, as, Vol. 6, Fig. 2, etc.

(*c*) Use numerals for all numbers exceeding twelve, with the following exceptions:

1. Do not begin a sentence with a numeral.

2. Round numbers used in an indefinite sense shall be spelled out; as, "A *hundred* feet or so," etc.

3. Numbers shall be spelled out when used in the following manner: "*fifteen* 1-in. rods," etc.

(*d*) In decimal numbers having no units, a cipher shall be placed before the decimal point: as, "0.65 in."; not ".65 in."

(*e*) In expressing percentages, precise figures, etc., use decimals: as, "4.5 per cent"; not "4½ per cent."

(*f*) Omit unnecessary ciphers in sums of money, as, "$3"; not "$3.00."

(*g*) In pointing off numbers of more than four figures, use commas in the text (1,234,567) and spaces in tabular matter (1.234 567). Numbers of four figures shall not be pointed off in either text or tabular matter (1234), except when they occur in a table containing any number of more than four figures.

(*h*) Always use numerals for the day of the month when the month is given (January 25, 1913) and for the time of day (2.30 p. m.).

Spelling and Punctuation.

8. *SPELLING AND PUNCTUATION.*

(*a*) *Simple Words.*—The following spelling shall be used:

aging
briquette
cent*er*
crystallin
dis*k*
*e*mbed
fib*er*
formulas
fulfil
gage
gasol*i*ne
*i*nsure
*m*old
oxid*e*
paraffin
program
re*i*nforced
skil*l*ful
sul*f*ur
turpentine

(*b*) *Compound Words.*—The following spelling shall be used:

Spell with hyphen.

cold-rolled
cross-section
one-half
open-hearth

Spell without hyphen when used as noun.

cast iron
plaster of Paris
testing machine
wrought iron

Spell as one word.

cooperate
eyebar
fireproof
footnote
quicklime
reroll
retest
reweigh

(*c*) Compound adjectives shall be hyphenated; as, "2-in. gage," "cast-iron cylinder." Such expressions as the following shall be written *without* the hyphen after the first numeral: "2 and 6-in. specimens."

Capitals.

9. *CAPITALS.*

(*a*) Use capitals sparingly.

(*b*) Capitalize the principal words in headings, titles of books, papers, etc. (nouns, verbs, adjectives and adverbs).

(*c*) Use capital initial "C" for "committee" when used as a title: thus, "Committee on Organization," "Committee on Papers." In all other cases use lower-case "c"; thus, "The committee recommends ......"

(*d*) Use capital initial "B" for Bessemer; lower-case "p" for portland.

(*e*) Use initial capitals in reference to volumes, figures, plates, etc.: as, Vol. 6, Fig. 2, Plate VI, Table III.

(*f*) Use the form "test No. 1," "specimen A," etc.

10. *STANDARD TERMS AND FORMS OF EXPRESSION.*

Standard Terms and Forms of Expression.

(*a*) Use "shall" wherever the standards are to be made binding on parties of the first or second part.

(*b*) Use "will" wherever the standards are intended to express a declaration of purpose not mandatory upon the parties of the first or second part.

(*c*) Use "may" wherever the standards provide definitely for alternative courses.

(*d*) Use "full-size tests"; not "full-size*d* tests," etc.

(*e*) Use "gage length"; not "gage*d* length."

(*f*) Use "test *specimen*" not "test *piece*." In case the term "test specimen" is repeated several times in the same section, the word "specimen" may be used after the first use of "test specimen."

(*g*) Use "$\frac{3}{8}$ in. *or* over in thickness"; not "$\frac{3}{8}$ in. *and* over."

(*h*) In referring to dimensions, use simply "2 in."; not "two inches (2 in.)," or "two (2) inches."

(*i*) Use the form "without *cracking*" in referring to bend tests of metals; not "without *sign of cracking*."

(*j*) Use "reduction of area"; not "reduction *in* area" or "*contraction* in area."

11. *FOOTNOTES.*

Footnotes.

(*a*) Use superior figures instead of asterisks, etc., except in connection with numerals, for which use letters.

(*b*) The names of journals, proceedings, bulletins, etc., shall be printed in italics, without quotation marks; the titles of papers and reports should be printed in Roman and enclosed in quotation marks.

(*c*) Abbreviate the names of societies.

(*d*) When reference is made to a paper or report by title, only the initial page should appear in the footnote. Thus:

(Name of Author)........, "........(Title of Paper)........," *Proceedings*, Am. Conc. Inst., Vol. XVI, p. 283 (1920).

When such titles are not given, or when reference is made to certain parts of papers, reports, etc., page numbers should be indicated, as follows:

Engineering News-Record, March 10, 1921, pp. 412–414.

(*e*) When volume numbers are given, the year of publication shall appear in parentheses at the end of the footnote. Otherwise, the date of publication should appear immediately after the name of publication. (See above examples.)

REPORT ON COMMITTEE E-3, ON RESEARCH

The work assigned to the committee is as follows:

1. Program of tests of concrete mixers.

2. Report on data available as to, and on what further investigation would be necessary to determine, the effect of admixtures in concrete.

3. Report on data available as to "surface area," "fineness modulus" and "water-ratio" theories in the design of concrete mixes.

4. Review and list important developments.

5. In conjunction with Committee on Reinforced Concrete, an investigation be outlined, partly laboratory and partly on construction work, to throw more light upon the selection of aggregates, the proportioning of concrete, the effect of time of mixing, effect of water content, etc., all of which will aid the committee and members of the Institute in passing upon the problem of specifying concrete by strength. On August 19 Mr. Whipple suggested that the A. C. I. Committee C-6, on Field Methods, N. M. Loney, chairman, is interested, and that Messrs. Ford (E-1), Loney (C-6), and Hatt (E-3), confer upon this problem.

Assignments of Sub-Committees:

1. Program of tests of concrete mixers assigned to sub-committee, Slater, Abrams and Hatt.

2. Admixtures in concrete; Abrams, Pearson and Hollister.

3. Theory of proportioning concrete; Talbot, Abrams and Young.

4. Account of current investigations; Pearson and Hatt.

5. The application of methods of control to field conditions; consistency tests; proportioning; making of field specimens; among other things to report upon the relation of strength of field cylinders to cores cut from the work; S. C. Hollister and A. R. Lord to represent Committee on Research in discussion of this topic with Committee C-6, on Field Methods.

1. Program of Test of Concrete Mixers:

The attached program of tests of concrete mixers was drafted by Prof. Abrams and discussed by correspondence. It is not supposed that a complete program such as this would be undertaken; but that the number of groups and items would be lessened when an investigation is placed under operation. Six different types of mixers are indicated by letters.

Group 1: Determines the effect of time of mixing, using pebbles as the coarse aggregate and a relative consistency of 1-10.

Group 2: Studies the action of the mixer on concrete of varying consistency.

Group 3: Studies the mixer action on various aggregates.

Group 4: Provides mixers of a selected type but of varying capacity.

Group 5: Studies the sequence of charging the various materials. So far as is known now the best results are obtained by an early admission of the water.

Group 6: Examines the action of continuous mixers of improved types.

The purpose of such an investigation is to determine the optimum conditions of operation of the mixer in producing well mixed concrete economically. Suggestions for improvement in individual mixers might be expected to result from the tests. The fundamental idea is not that of a horse race.

The investigation should be at a central point where uniform aggregate could be stored and kept at constant moisture content; and from which the concrete could be taken in motor trucks to a point of useful construction, from which cores could be cut later.

The mixer should be electrically operated and measurements of power input made.

Such an investigation requires a large operating force of workmen, observers and other technical assistants, whose time-program would be economically disposed when several mixers are under test one after the other. The various mixers should be interspersed in the program of operation.

It is believed that the entire contents of the mixer should be discharged at once, and that the mixer should not be robbed of samples during a run. At least two batches of concrete should be mixed for any one condition on the program.

2. *Admixtures with Concrete:*

The committee has no report to make upon this topic.

3. *Theory of Proportioning Concrete:*

Some discussion by correspondence has been had concerning the theory of proportioning concrete mixes. It is to be hoped that a review of the present knowledge upon this subject will determine what elements will be truly predictive of the strength of the mixed concrete. Such investigations should also include the examination into the success with which the various methods have been attended when applied commercially, the difficulties encountered in their use and the modifications, if any, required to make them practical; also the effect of these on the cost of the work.

It is hoped that this subject will be continued next year and the sub-committee instructed to bring in a report for the 1923 meeting.

4. Account of Current Investigations:

There is appended to this report a list of investigations which are current insofar as the committee has been able to learn of these through correspondence. This list is presented as a matter of information for the use of the various committees of the American Concrete Institute, who must form their conclusions upon the basis of scientifically determined data.

Attention is called to a more thorough census of research and research works which is now in operation through the National Research Council and the Bureau of Public Roads.

General Comment on Research.

Perhaps it will be useful to define what is included under the term "research." The following paragraphs are advanced in the absence of more perfect definitions:

Discussion of Qualities of Research:

1. Research work may be in industrial fields with a definite aim or in pure science without thought of particular use.

2. The research may lie in the field of any branch of knowledge that is treated in a scientific spirit, including, of course, the natural sciences, but not excluding mathematics, history, economics and medicine.

3. The method may be by logical deduction, based upon natural phenomena, and always referenced thereto, or by induction using the methods of modern experimental science, both to discover new facts or uncover underlying laws, or the determination of the mechanism by which one event follows another, resulting in a substantial addition to the existing body of knowledge.

4. The qualities by which true research are known are, earnestness, devotion, diligence and system, and, in highest form, original and creative work.

5. Recognition should be given to skill and initiative in devising suitable methods and apparatus for use in observations, and care and devotion shown in difficult observations, skill in arranging and interpreting data, or forming generalizations upon them.

6. The result should be the discovery of something hitherto unknown.

The following are activities and workers not considered as research.

1. The work of the mere routine workers without initiative or responsibility.
2. The manual operations of the mere mechanic.
3. The work of the merely critical and negative mind.
4. Purely regulatory functions.

5. Those with only an amateurish and discontinuous interest in research.

6. Those with only the acquisitions of the learned scholar, who may have absorbed the results of research.

7. Inventors, unless they are scientists.

RECENT DEVELOPMENTS.

A growing interest in research is evidenced. The Advisory Board on Highway Research of the Division of Engineering of the National Research Council, has secured a director who has been active since July studying the field, organizing a general program of highway research, and making contact with the highway commissions of the several states and the other fruitful sources of research data. The term "highway research" indicates that the program includes not only road construction but the traffic and its relation to the road in both mechanical and economical relations.

The Society of Automotive Engineers, which represents the technical branch of the automobile and truck industry, has established a research department under the direction of Dr. H. C. Dickinson, formerly of the Bureau of Standards.

Especial mention should be made of the research activities of the Bureau of Public Roads. The magazine "Public Roads," published by the bureau, was rapidly becoming a magazine of highway research when a blanket order of Congress temporarily discontinued its publication.

The recently organized committee on research of the Am. Soc. C. E., and the Western Society of Engineers, give promise of increased activity in the study of most effective channels by which research will come to useful application, and workers will be stimulated and supported.

The training of selected research talent is a crying need; for if all the research projects of undoubted value are adequately financed, the personnel to conduct these projects would be wanting. Recognition of the special function of research in state and federal organizations, in industry and in the universities, and a provision for compensation which will attract men of research talent, will in time build up a personnel. This word research occurring now so frequently in the general and technical press, indicates the education of the public to the value of that form of acitivity. Particularly in the highway field, it is clear that we should as soon as possible come out from the field of opinion and bring highway engineering and highway operation to the basis of scientifically determined data.

It is needless to comment upon the economics to be effected by research. At even fractional percentages the dividends will be enormous. The money to be spent in highway construction in the United States in the next few years is somewhere between one and two billions of dollars, and a much larger sum of costs of operation in the inefficient motor vehicles that travel over them. In comparison the sums spent in research are meager. This is well recognized. What is needed now is a drive to translate the will for research into action in the form of appropriations, and to train research workers.

The best center of information for the research workers as concerns their common interests appear to be in the National Research Council. Additional organizations and committees for research should orient themselves to the purposes of this central clearing house for national and international research.

Attention is called to three notable instances of research on a large scale, namely, the Bates road near Springfield, Illinois, the Pittsburgh test road north of San Francisco, Cal., and the investigations of the Bureau of Public Roads. These three investigations probably represent an expenditure in construction and operating costs of nearly one-half million dollars to date. These investigations have each a definite aim through scientific procedure.

STRUCTURE OF COMMITTEE ORGANIZATIONS.

The new interest in research is not unattended by complications with respect to a growing complexity of committee organization. There is much to be gained when individual researchers work independently on the same problem, but only a loss when committees duplicate their functions. A confusion only exists when various committees of different organizations, or in the same organization, send out questionnaires seeking information and inquire into the progress of research. The individual research worker may be bewildered by the independent approach of so many different committees, and pray to be protected from his friends.

It is suggested that a useful task would be performed if a census be made of the various committees of the several technical societies to determine the distribution of personnel and the overlapping fields of effort. The organizations concerned would probably include the American Society of Testing Materials, the American Railway Engineering Association, American Association of State Highway Officials, American Concrete Institute, American Society of Civil Engineers, and committees of the Advisory Board on Highway Research.

RESEARCHES RECOMMENDED FOR EARLY PROSECUTION.

The committee has received from various sources suggestions for those research projects which seem to be of immediate importance, and these have been listed below:

Structures (Submitted by Mr. W. A. Slater):

1. Bond between steel and concrete.
    - (a) Tests of beams so designed that failure will be by bond and so laid as to indicate the upper limits.
    - (b) Tests laid out in such a way as to determine the characteristics of a proper deformed bar.
2. Effectiveness of various methods of anchoring reinforcing bars.
3. Further tests of columns to determine as to:

   (a) Effectiveness of various percentages (especially large percentages) of longitudinal reinforcement in combination with various percentages of spiral.
   (b) Effect of long continued load (especially under conditions to bring out effect of shrinkage) on distribution of stress between steel and concrete and on ultimate strength of columns.
   (c) Effect of introducing varying known amounts of bending moment into columns under (a) and (b).
4. Suitability of various grades of steel (structural, intermediate, hard, and rerolled bars) as reinforcement together with a study of the proper methods of testing to determine specifications for reinforcement.
5. Tests of slabs supported on beams on four sides with and without intermediate girders. This field should be investigated as fully as the flat slab has been investigated and should be approached more methodically than the flat slab was approached. We believe this investigation can be carried out at much less expense than has been the case with the flat slab.
6. A study of the behavior of columns in structures in which the slabs carried by the columns are loaded so as to put heavy, direct and bending loads on the columns.
7. Investigation of loads actually in place on a large number of floors together with loads for which the same buildings were designed.
8. Determination of relative effectiveness, in producing stress, of loads applied definitely at points as in laboratory tests and of loads applied by piling various kinds of loading materials upon a floor.
9. Slabs supported on four sides built,
   (a) Of reinforced concrete alone;
   (b) Of tiles and concrete ribs.

Materials:

1. Effect of very fine particles such as limestone dust in aggregates on the strength and permanence of concrete.
2. Engineering and economic relation between pit-run gravel and plant-produced aggregates.
3. The design of deformed bars.
4. The relative value of structural and high carbon steel for structural and temperature reinforcing.
5. The manufacture of blast furnace slags to yield improved aggregates.

Testing and Design of Materials:

1. The study of methods for determining consistency, including calibration and standardizing of the flow table.

2. Development of a hardness test for gravel aggregates.
3. Methods of field testing of concrete, especially the development of a small sized suitable testing machine.
4. An index of the quality of aggregates. Can the proportions of the material determined from mechanical analysis be designed to produce concrete of desired workability and strength without actual strength tests?
5. Effect of water cement ratio, gradation of aggregate, etc., on properties other than the compression test.
6. The analysis of existing data to determine what further test need to be made for determining the validity of the several theories advanced for the design of concrete mixtures such as fineness schedule, surface area and voids.
7. A study of field conditions to determine the extent to which precise laboratory control of moisture, gradation of aggregate and laboratory technique can be duplicated in the field. In other words, the degree to which construction work can be regulated to secure uniformity of the manufactured concrete.
8. The use and means of improvement of low grade aggregates.
9. The strength of easily flowing concrete determined from cores bored from the finished work.
10. The value of integral water-proofing substances, as shown by the quality of cores bored from construction made under field conditions.
11. Effect of hardeners and accelerators on the properties of mortar and concrete.
12. Study of the most economic methods of curing concrete.
13. Test of concrete under repeated and reversed stresses to determine the per cent of ultimate stress that can be applied indefinitely.
14. Study of the optimum condition of operation of various concrete mixers, and of the improvements in the action of machine mixers.
16. Methods of giving added plasticity to Portland cement for interior use in plastering, etc.

Design:

1. Study of the design of plain concrete road slabs of different thicknesses and with inverted curbs, etc.
2. Study of the manufacture and use of precast slabs.
3. Study of the theory and practice of reinforcement of concrete road slabs, including the economics of construction and maintenance.
4. The implications of the plastic flow of set concrete upon the design of reinforced-concrete structures.

5. Development of the most economic form of slab floor for concrete houses and specification for the minimum floor load which at present is often fixed in construction by the stiffness of the wood floor construction.

W. K. HATT, *Chairman.*

OUTLINE OF PROPOSED TESTS OF CONCRETE MIXERS.

(American Concrete Institute, 1922)

2 runs for each condition. Mixer type denoted by letters A, B, C, etc.

The same cement and fine aggregate to be used in all tests.

All concrete to be discharged from mixer at the periods shown.

Test cylinders to be molded in accordance with "Tentative Methods of Making Compression Tests of Concrete" of the American Society for Testing Materials.

OUTLINE BASED ON ABRAMS' PLAN.

Group	Mixer.	Capacity, cu. ft.	Time of Mixing, min.	Mix.	Coarse Aggregate.	Relative Consistency.	Age at Test.	Cylinders in a Set.	Batches for each Mixer.	Total Batches.
1*	A B C D E F	14	¼ ½ ¾ 1 2 5 10	1–7 1–6 1–5 1–4 1–3½ 1–3	Pebbles	1.10	7 d. 28 d. 3 m. 1 y. 3 y.	3	48	96
2*	A B C D E F	14	¼ ½ ¾ 1 2 5 10		Pebbles	0.90 1.00 1.10 1.25 1.50 2.00	7 d. 28 d. 3 m. 1 y. 3 y.	3	40	240
3*	A B C D E F	14	¼ ½ ¾ 1 2 5 10	1–4	Limestone Trap Slag Granite	1.10	7 d. 28 d. 3 m. 1 y. 3 y.	3	20	60
4*	C†	4 9 21 28		1–5	Pebbles	1.10	7 d. 28 d. 3 m. 1 y. 3 y.	2‡	2	8
5	A B	14	Study of sequence of charging batch mixers. Details to be worked out.							
6	G H	Continuous Mixers		1–5	Pebbles	1.10	7 d. 28 d. 3 m. 1 y. 3 y.	2‡		Details to be worked out.

* Groups 1 to 4 are based on the assumption that 6 types of mixers will be used.

† From one manufacturer secure mixers of a graduated series of capacities.

‡ Duplicate batches to be mixed on different days.

APPENDIX TO REPORT OF COMMITTEE ON RESEARCH, AMERICAN CONCRETE INSTITUTE, 1922.

CURRENT CONCRETE RESEARCH PROJECTS.

(Incomplete Census.)

University of Minnesota:

Investigations in concrete carried on by the University of Minnesota are being conducted by Professors Maney and M. B. Lagaard in the structural division of the civil department, and by Prof. F. C. Lang, of the highway division. The major part of the work, begun by Mr. MacMillan, is still being carried on at the university. The series of tests include:

1. A study of shrinkage in plain concrete over periods up to 8 years. These tests cover the study of effect of variations in (a) preparations using neat cement mixtures and concrete of usual mixes; (b) curing, such as dry air with variations in temperature, moist air, with water and steam cured; (c) kinds of coarse aggregate; (d) shape of specimens and direction and position of gage line measurements; (e) per cent of water used; (f) density of concrete; (g) surface tension of water used; (h) foreign ingredients, such as hydrated lime, plaster of Paris, etc. (To be published in final form this year.)
2. A study of shrinkage and time yield or plastic flow of concrete under sustained loads over long periods of time. These include measurements on reinforced concrete bridges, buildings, and concrete chimneys under actual service conditions, as well as on laboratory slabs and beams. (To be published in final form this year.)
3. Load extensometer tests of a one-half size model highway bridge of cellular slab construction.

   Complete data was secured regarding the shrinkage of the concrete and the conditions of the slab previous to the load, covering a period of one week when bridge was tested to failure. (Test completed several years back but date of publication uncertain.)

In addition to the above tests there are being made:

1. A comparison of the important methods now proposed for designing concrete mixtures. A number of tests made at present point to certain conclusions but do not justify print owing to the magnitude of the problem.
2. A study of steam curing under pressure as a means of obtaining the requirements of the 28-day test limit.
3. A special study of reinforced-concrete columns to let additional light on the behavior of columns with particular reference to the

method of designing proposed by Mr. MacMillan at the last meeting of the American Concrete Institute.

These tests are being conducted under the advice of Mr. MacMillan and the results should prove to be of particular interest. The work will be completed this year.

The Highway Division has undertaken:

1. A study of the expansion and contraction of concrete specimens cut from pavements throughout the state due to temperature changes and to moisture conditions.
2. A series of tests on concrete made from aggregate containing shale, the concrete being exposed to alternate freezing and thawing for periods up to six months. The object is to obtain effect on the wearing surfaces as well as the strength with the shale present.

   There is also under consideration a study of the structural behavior of road slabs with various types of reinforcements, it being proposed to select a certain section of state road in which are to be constructed various test sections for investigational purposes.

Lafayette College, Easton, Pa.

The following investigations are being carried on at Lafayette College under the direction of Prof. F. O. Dufour:

1. To find a criterion for determining when fine aggregates proportioned in accordance with the requirements of Abrams' design theory for concrete mixtures, will not meet the 28-day strength requirements.

   When Professor Dufour was with Stone & Webber, 40 tests were made with a fine sand and the aggregates properly proportioned according to Abrams' theory, but there resulted not more than 1600 lb. per sq. in. at 28 days. A second sand proportioned according to Abrams' method gave 2000 lb. per sq. in. It is expected that by next fall there will be available results from 100 additional tests on 25 different sands.
2. To determine the effect of vibrations during the time of setting on the strength of concrete.

   In this connection a series of ten tests were made by Pennsylvania R. R. men on mortar beams not reinforced. These tests are being continued and extended to both plain and reinforced concrete beams. The vibration is to be produced by means of pneumatic hammers. This is an accellerated test, but when weather permits the specimens will be poured in place on a railroad bridge and left to the vibrations of passing trains. These tests will be carried on during the next eight months.

3. To determine the effect of variations from these required by specifications in the size of screen openings, and the percentage of sizes and the strength of concrete.

   Details for this investigation have not been decided upon. It is sometimes the case that carloads of stone shipped from a great distance are rejected by inspectors when 11½ or 12 per cent pass the ¼ in. screen instead of 10 or 11 per cent required by specifications. Also, inspectors will reject stone at the plant when the screen becomes a little worn, causing the holes to be slightly larger than required. The question is whether a variation in the strength of the concrete is greater or less than the variation between the single specimens made from stone which meets the requirements of the specifications.

University of Illinois:

The following investigations in concrete are being carried on at the University of Illinois, under the direction of Prof. A. N. Talbot:

1. Reinforced-concrete arch test.

   This test was undertaken to determine the relation between actual and computed stresses. It involves the correctness of the assumption usually made in a theoretical analysis.
2. Reinforced-concrete columns.

   The purpose of this investigation is to find the action of spiral reinforcement and the stresses produced in it; the effect of pitch, of spiral, of length of column, of eccentricity, of richness of concrete, and the combinations with longitudinal reinforcements; also the frictional resistance of granular materials restrained by hooping.
3. Investigation of web strength of beams. (Web strength used instead of shearing strength.)
4. Investigation of cantilever flat slabs.
5. A bulletin on the elements entering into the strength of concrete is now being prepared.

Lehigh University, Bethlehem, Penna.

The following extensive series of tests were outlined and carried out under the direction of A. R. Lord, at Fritz Engineering Laboratory, Lehigh University.

1. Pull-out tests of straight and hooked bars imbedded in concrete specimens.

   Data are in possession of Civil Engineering Department, Lehigh University (Prof. Ralph J. Fogg in charge).

Iowa State College, Ames, Iowa.

Under the direction of Dean Anson Marston and Prof. T. R. Agg.

1. Load construction and steel floor joists of wood floor highway bridges. Bulletin No. 53, published in 1919.
2. Comparative cement tests—continuous investigation carried on to check work of station with similar work done at other stations.
3. Chemical determination of cement content of concrete.
4. A study of the load carrying properties of precast pipe culverts.
5. Freezing and thawing tests of clay and concrete drain tile.
6. Proportioning pit run gravel concrete.

   This investigation has been completed.
7. Investigation of wear on concrete limestone aggregates. A study to determine the advisability of using Iowa limestone in concrete pavement construction. Published soon.
8. The effect of alkali on concrete. A study of the factors affecting the durability of concrete drain tile when laid in alkali soil.

Pennsylvania State College, State College, Penn.

Under the direction of Dean R. L. Sackett.

1. A method for determining when the form should be removed from a concrete wall.

Purdue University, Lafayette, Ind.

Under the direction of Prof. W. K. Hatt.

1. Study of the effects of reversal stresses on plain concrete specimens, to determine fatigue of concrete. Progress report.
2. Study of the ball test applied to the surface of mortar slabs. A new and useful test for concrete. Progress report.
3. Study of the effect of so-called tolerance material on coarse aggregate. Complete and published.
4. A survey of the road materials of Indiana.

Kansas State Agricultural College, Manhattan, Kan.

R. A. Seaton, Dean, Division of Engineering.

1. Study of temperature stresses on rigid pavement slabs.

   Determination of the actual concentration and expansion due to temperature and other causes and the stresses induced thereby in concrete and brick pavements of various types and various kinds of sub-grade.
2. Wear test of concrete.

   It is proposed to devise a satisfactory wear test investigation of concrete not involving the addition of expensive equipment.
3. Concrete used in highway construction.

   Field specimens of concrete are taken during construction and sent to the laboratory for testing. Data covering the use of a wide range of aggregates and proportions covering pit-run gravel are included.

Oregon State Agricultural College, Corvallis, Ore.

1. Investigation of concrete sands of Oregon.

University of Maryland, College Park, Md.

Under the direction of Dean A. N. Johnson.

1. Effect on character on concrete in Maryland roads due to traffic and climatic changes.
2. Fatigue test on plain concrete. New apparatus in which an eccentric shaft revolves between two beams.

U. S. Bureau of Public Roads, Washington, D. C.

1. Determination of most suitable abrasion test for concrete (proposed).
2. Study of wear resisting properties of concrete aggregates.
   It is desired to determine (a) relations which exist between the wear and strength of concrete; (b) whether the same relations exist between the wear of concrete specimens in which dissimilar aggregates are used as are indicated by laboratory tests on the aggregates themselves; and (c) relation which exists between the wear and strength of concrete specimens in which the maximum size of coarse aggregate varies. (In progress.) Laboratory investigation.
3. Study of relation between wear and strength of concrete as prepared in laboratory, and results when used in construction.

   In this investigation behavior of specimens prepared in the laboratory is compared with similar specimens made and cured under the same conditions as the pavement. (Summer of 1922.)
4. Study of the effect of admixtures of various materials in concrete.

   In this project it is proposed to study the effect of admixtures of various substances with Portland cement for the purpose of (a) accelerating the strength; (b) preventing premature drying out of concrete; and (c) increasing workability. (In progress.) Laboratory investigation.
5. Detecting and neutralizing deleterious substances in concrete aggregates.

   It is proposed to make a detection of these substances which are of such nature as to prove harmful to cement mixtures, a quick and simple operation. The presence of organic matter as determined by present methods does not indicate the effect it will have on concrete with any certainty whatsoever. Mineral salts are seldom given consideration and little is known about the quality of these substances that would prove injurious. (In progress.)
6. Study of laboratory tests for determining quality of aggregates used in concretes.

   It is proposed to develop laboratory tests for determining quality of aggregates for use in concrete, that is, stone, gravel and sand. Tests as made at present have not been standardized and can be modified to advantage. (Proposed.)

7. Study of proposed compression test for cement.

   It is proposed to study the compression test for cement as a possible substitute for the standard tension test as now made. (In progress.)

8. The effects on character of concrete due to traffic and climatic changes.

   Samples which will represent a concrete that has been in use from one to ten years will be taken from the road by means of a core drill and subjected to laboratory investigation to determine their relative elastic qualities. (In cooperation with the University of Maryland.) Report being prepared by Dean A. H. Johnson.

9. Investigation of central plant mixed concrete. (Proposed.)

   It is proposed to determine a safe maximum time of haul of central plant mixed concrete.

10. Studies in treatment of concrete drain tile to prevent alkali action. (In progress.)

11. Study of blast furnace slags for use in concrete.

    It is proposed to study the concrete making properties of blast furnace slags in a large number of producing plants. (In progress.)

12. Investigation of the merits of reinforcements and special joints proposed and now used for distributing loads across joints or cracks and rigid pavements. (Proposed.)

13. Study of resistance of road slabs with artificial supports when subjected to static loads. (Proposed.)

14. Impact test on plain and reinforced concrete slabs.

    About eighty 7′ x 7′ specimens will be tested to failure by a road impact machine. This should throw light on the relative resistance afforded by different kinds of reinforcements to such impacts as can be expected from moving motor truck wheels. Report of first series of tests published October, 1921.

15. Abrasion test on concrete pavement sections. Report published in May, 1921.

IN ADDITION TO THE FOREGOING PROJECTS THE FOLLOWING INVESTIGATIONS SHOULD BE NOTED.

Illinois Department of Public Works
and Buildigs, Division of Highways.
Clifford Older, Chief Highway Engineer.

1. Investigational work in pavements and sub-grade being conducted on the Bates experimental road which is located 12 miles southwest of Springfield. This road which is over 2 miles long, is composed of 63 sections, each section differing in kind of pavement.

During the coming spring the road will be subjected to regular traffic which will vary from light loads to extremely heavy ones, and it will thus be possible to observe the resistance offered by different kinds of pavements. Preliminary tests on each of the sections have been made during the past year. This work has been carried on in cooperation with the U. S. Bureau of Public Roads. These experiments include:

1. Repeated loading on all types of pavements under summer temperature and conditions.
2. Repeated loading on all types of pavements under winter temperature and conditions.
3. Repeated loading on all types of concrete sections with sub-grade removed from under point of load application.
4. Action of pavements under changes of temperature.
5. Survey of cracks in pavements in relation to temperature.
6. Distribution to the sub-grade of loads on different pavement sections.

2. Investigational work in materials of construction is carried on in the laboratory at Springfield. Work in concrete naturally occupies the greatest amount of time. The principal experiment at present is the investigation of fatigue of concrete. This experiment deals with the failure of concrete after being subjected to many applications of a load less than the maximum which could be expected if it were applied statically. The following experiments have been conducted during the past year:

1. Curing of concrete, involving the use of 12 different methods on about 400 specimens.
2. Curing of concrete under low temperatures.
3. Prevention of bond in distributing bars in concrete roads.
4. Testing of patented materials such as earthcrete, supercement, etc.

Pittsburg, Cal., test road:

1500 ft. long, oval in shape, built by the Columbia Steel Co., at Pittsburg, Cal., under the engineering direction of Messrs. Lloyd & Aldrich, of San Francisco, with the cooperation of the Bureau of Public Roads and the California Highway Commission. Contributions have been made by the California automobile clubs.

The construction includes concrete slabs of various thicknesses and shape, some with inverted curbs and variously reinforced with steel, both carbon and open hearth running from 20 to 69 tons per mile.

This road was built July, 1920, and was first loaded Nov. 29, 1920. There were used 40 trucks equivalent to the passage of 10,000 trucks per day. Complete measurements of volume changes due to temperature and provision for sub-soil moisture. Deflections of slabs were recorded autographically in 4 tunnels $3\frac{1}{2}$ ft. under the road. Mechanical action of the slabs were determined by various positions of truck under static and impact conditions.

Casper, Wyoming.

Reinforced-concrete precast slab road, built in 1920, 2400 ft. long. Units are 8 ft. long, 9 ft. wide and 6 in. thick, plane and warped transverse joints.

Alexander County, Virginia.

Experimental concrete highway, 2 miles long, constructed under the direction of the U. S. Bureau of Public Roads.

Sections of various thicknesses and shapes and various reinforcements.

State Highway Department of Pennsylvania.

Under the direction of H. S. Mattimore.

1. Determination of relative strength of cores taken from the road and cylinders made during the construction of road.
2. Series of tests of acceleration, calcium chloride, vitriflux, Master Builders, Livingstone.
3. Determination of the effect of coal on the strength of sands. From 2 to 10 per cent of bituminous, anthracite and lignite were used.
4. Wearing value of sand in mortar specimens under Dorry test.
5. Relation of fineness modulus surface area to wear.
6. Test of absorption, using slag, sand stone and lime stone aggregates.
7. Tests for determining causes of hair cracking in concrete pavements.
8. Effect of hot water and hot cement used in summer upon the setting time of cement.
9. Impact test upon concrete cylinders.
10. Experimental road 6000 ft. long containing various mixtures of concrete with various aggregates and finishes.

University of Wisconsin Madison, Wis.

Under the direction of Prof. M. M. Withey.

1. Test of concrete slabs poured in pavements, compared with specimens made in the laboratory.

   Additional studies will be supplied by Professor Withey.

Wisconsin State Highway Commission:

1. Experimental road near Milwaukee, constructed by the Wisconsin Highway Commission, 1917, under the direction of H. J. Kuelling.

   This road contains both plain and reinforced concrete sections, reinforcements consisting of Clinton wire, rib metal, and $\frac{1}{2}$ to $\frac{5}{8}$ in. rods.

The last inspection on this work was made July 7-8, 1920. The general observations were:

1. Reinforcements as now placed by the different states, and even heavier, cannot be recognized as a factor in preventing cracks. It can only be considered, as far as the crack situation is concerned, as a means of holding the cracked sections together.
2. That as far as cracking of the road is concerned, due to weather, the majority of reinforcement, if not all of it, should be placed across the road.

Bureau of Highways, City of Philadelphia:

On the Byberry-Ben Salem Pike.

A large number of sections were constructed on that highway about 1912. This work was done so many years ago that it is not representative of modern practice, especially in regard to the concrete paving sections.

Laboratory of the Hydro-Electric Power Commission of Ontario.

R. B. Young, Assistant Laboratory Engineer.

1. Design of concrete mixtures covering the correlation and field application of different theories of proportioning.
2. Testing of fine aggregates both acceptance and performance testing, particularly methods for quickly determining quantitatively the relative cost at a common point of materials of different characteristics, from different sources and of varying cost.
3. The economics of concrete mixtures—the effect on cost of different sizes and combinations of aggregates giving concretes of equal workability and strength.
4. Workability or consistency of concrete mixtures. Methods of determination.
5. Control of quality in the field and the steps necessary to obtain it.

University of Texas, Austin, Texas:

There is being carried on at the University of Texas, under the direction of Prof. F. E. Giesecke, an investigation for the determination of concrete to abrasion.

Specimens of concrete are subjected to abrasion produced by a specially constructed apparatus which is intended to simulate the conditions in a road.

Bureau of Standards, Washington, D. C.

1. Study of surface treatment of concrete by waterproofing materials, when necessary, to prevent deterioration.
2. Value of various coatings to protect concrete tanks against leakage of light oils.
3. Study of concrete exposed to action of alkali and seawater.

The following are titles of investigations reported by W. A. Slater, the data of which are on file in the Bureau of Standards or elsewhere, but which have not been published:

1. Tests of shearing strength of concrete beams; a large and valuable investigation; report 75 per cent complete.

2. Tests to determine relative values of expanded metal and of loose bars (placed parallel to and at 45° with the direction of the span) as reinforcement for slabs supported on two sides; results not conclusive but of considerable value; report 100 per cent complete.
3. Tests of U frames having brackets of various proportions. Report aims at arriving at a simplified semi-analytical method of applying the results to the design of frame structures; should be of much value; report 75 per cent complete.
4. Impact tests of reinforced-concrete slabs; report 75 per cent complete.
5. Leakage tests through cracks in hollow concrete beams; water inside of beams held under heads varying from zero to 30 feet; report 50 per cent complete.
6. Investigation of causes of cracking in a concrete barge; report 85 per cent complete.

Department of Civil Engineering, University of Illinois, Urbana, Ill.

Investigation of the magnitude, cause and effect of warping of road slabs. Slabs and apparatus have been installed at Urbana, Illinois, and daily readings taken since the middle of December.

Ctructural Materials Research Laboratory,
Lewis Institute, Chicago, Illinois.
Under the direction of Prof. Duff A. Abrams:

1. Tests of concrete exposed to alkali.

   About 1800 test blocks 10 x 24 in. in diameter were installed at points in Colorado, South Dakota, and Western Canada, where alkali solutions are prevalent. About 6000 smaller specimens have been made for exposure at the laboratory to different alkali solutions at different concentrations.
2. Tests of accelerators in concrete.

   A comprehensive investigation of calcium chloride and other accelerators is being carried out on concrete, using different mixtures, consistencies, etc., and making tests at ages from 2 days to 3 years.

   In one series of tests different brands of cement are being used and the curing condition is being carried over a wide range.

3. Concrete-making properties of fine aggregates.

   About 200 different samples of sand and other fine aggregates are being used in the study of the wear and compressive strength of concrete.

4. Concrete-making properties of coarse aggregates.

   About 150 different samples of coarse aggregates were used in an investigation of the compressive strength and wearing resistance of concrete. Parallel abrasion tests are being made on the aggregates.

5. Transverse strength of concrete.

   A report on transverse tests of concrete including the three-months' period will be presented to the American Concrete Institute at Cleveland next week.

6. Use of impure waters for mixing concrete.

   Two series of investigations have been carried out, using a large number of samples of impure waters from various sources, including sea water, alkali waters, mine water, bog waters, waters from mineral springs, sewage and various industrial wastes.

7. Tests of bin-stored cement.

   A 2000-bbl. bin of portland cement has been set aside by one of the cement manufacturers for a study of the effect of storing cement in bulk for long periods of time. Parallel tests are being made on samples stored under the following conditions:

   1. In cloth sacks under tarpaulin in yard.
   2. In cloth sacks in shed in yard.
   3. In cloth sacks in basement of Lewis Institute.
   4. In wooden barrels in shed in yard.

   Samples for test will be taken at intervals up to 2 or 3 years.

8. Tests of large-size aggregates.

   Samples of pebbles, crushed granits, limestone, trap and slag, graded up to 3 to 4 in., will be used in this investigation.

9. Effect of shape of test piece on the compressive strength of concrete.

   Tests will be made on concrete cylinders, prisms and cubes of different dimensions, using a wide range of mixtures, consistencies, etc. Tests will be made at ages varying from 7 days to 1 year.

DISCUSSION.

Ernest Ashton.—Would not a well directed publicity campaign directed to the industrial businessmen and bankers produce sympathetic appreciation of the necessity of research work? Mr. Ashton.

W. K. Hatt.—It has been part of my duty recently to act as traveling salesman of research in connection with highway construction and operation. The highway officials are now pretty well convinced that the money of the people can not be spent wisely in building roads unless on a basis of facts secured through scientific research. The next thing to do is to persuade these gentlemen to create a research staff, men with a special function, as Mr. Older has done in the State of Illinois. But he has some 80 men in his testing and research staff; 10 of those are research workers pure and simple. Mr. Mattimore, in the State of Pennsylvania, has 65 people on his testing staff and controlling material. Now, if we can get the State Highway Commissions to recognize the research function, we will be a long way on, because these men have resources that need to be unlocked. Mr. Hatt.

H. C. Turner.—The research work that has been done has been done very largely by the industries themselves. Take the steel industry. The research has been done by the larger steel companies appropriating money for research work, and the same thing is true in the chemical line and in the Portland Cement Association. It seems to me that the Portland Cement Association has pointed the way in research work in cement. I think they have done wonderful work. It has been hoped by many in this convention that we might plan some definite research work to be directed by the Institute, and the advantages to be obtained through the contractors, through the companies supplying material, like steel and possibly machinery, essential industries in connection with the concrete industry, and possibly through some assistance from the Portland Cement Association and in co-operation with the Lewis Institute and with the Bureau of Standards in Washington. Mr. Turner.

You recall that our research work has been directed very largely by the experiment station at the University of Illinois, by Purdue University, and by work at the Bureau of Standards. It ought to be possible to bring together those who are actively in the business, to provide certain money and then co-ordinate the use of facilities and experts in these particular agencies.

It is not practical to expect our engineers or architects to make any large contribution, financially; contributions must come from the corporations and partnership members of the Institute.

A. E. Lindau.—Research work to be carried on effectively, of course, requires a considerable amount of money. Dr. Hatt has mentioned three projects which require nearly half a million dollars and these three projects merely scratch the surface. Private corporations have research departments now. A very notable example is the General Electric Com- Mr. Lindau.

Mr. Lindau. pany, but it is not a simple matter for a corporation to go into research work. The corporation must be of a rather high-minded type and be willing to take its dividends at the end of 10 or 20 years instead of 12 months. It is a long game to get dividends from research and it is something in the nature of public work as well. One source of income that may be tapped, of course, might be funds collected from trade associations. The status of these associations at the present time, however, is rather clouded and they are not falling over themselves, so to speak, to get together for a purpose of this kind, or in fact any other purpose. But trade associations, I think, should be encouraged to contribute funds for research work. In fact, this is being done now by some associations in a small way.

Mr. Brown. H. WHITTEMORE BROWN.—The speaker's experience as research engineer for a company interested in industrial housing leads him to the conclusion that research work of lasting value cannot and should not be done by commercial agencies. Research work undertaken by an organization primarily devoted to the making of profits is bound to be tinged by the necessity of a fairly quick return for the money expended. While this research work may be generally valuable, its value to the community as a whole is limited by the use which the company makes of it.

Research work which will be of greatest benefit to all can only be done by some central agency which is not organized for monetary gains. Such an organization exists at the present time in the form of the Bureau of Standards at Washington. The speaker feels that the proper method of increasing the quantity and quality of essential research work in this country should be by interesting congressmen and others in the work of the Bureau of Standards, to the end that larger appropriations may be given to this Bureau, thus allowing them to increase the scope of their work..

Mr. Hatt. PROF. HATT.—The tests of metals now being conducted under the auspices of the University of Illinois can be instanced. I do not know how much money is being spent. I know one steel company that put $40,000 into that research, for if Prof. Moore doesn't solve this problem we must do it ourselves. This is an example of a splendid research financed by industrial organizations that apparently know that that work is going to be of value. 'We need not say that funds must come only from taxes. It so happens that our states have very large sums of money for road construction, providing also frozen assets of research that might be liquidated to the advantage of the people. The industries also have large assets of research that ought to be unlocked.

Mr. Ashton. ERNEST ASHTON.—I would like to reply to Mr. Brown with regard to relying on Government activities for research work. The Bureau of Standards are wonderfully located, with a fine staff of research men, the finest equipped laboratories in the country. The appropriation comes up before Congress; it is pruned; it is stopped; our congressmen are not sold on the idea of research; they do not know what the Bureau of Standards is for; they are not particularly interested because the industrial organizations in their districts have not acquainted them with its value. You can talk to your own congressman and I question whether he has ever been out to see it. One trip out there and he would be sold. We have got to sell the people, the capitalists, the businessmen of this country, on the idea of research. There is one thing we can do, and that is to talk around the country, wherever we meet a fellow who might have any influence, tell him to ask his congressmen about using this investment at the Bureau of Standards. **Mr. Ashton.**

H. C. TURNER.—The President has received several letters from the Bureau of Standards in Washington offering its assistance in co-operation with the Institute, and only this week received a letter in which the hope was expressed that following this convention we might have a session between some committee of the Institute with the Bureau, in planning out research work of the character thought of primary importance by the members of the Institute. Director Stratton has assured us of their great desire to co-operate and aid in any work we wish to carry out. If we are going to make progress, I think we have got to make it through obtaining funds from individuals and corporations actively in the business. If we attempt to persuade congress to appropriate money, we will have to wait a number of years. I think the only safe procedure is to bring home to ourselves the great importance of the subject and collect money among ourselves and then carry on a program of research work in co-operation with the government and with the Portland Cement Association. **Mr. Turner.**

D. S. HUMPHREY.—I should like to ask Dr. Hatt for a definite recommendation, what he thinks the Institute can do as their share of the research work, the particular things he might have in mind that we are to do. **Mr. Humphrey.**

MR. HATT.—As the Committee report states, I would recommend that the proper committees of the Institute interest the mixer manufacturers in a study of the operations of concrete mixers. I would also recommend investigations relating to the strength of building floors, as suggested by Mr. Slater. This is a kind of work that cannot be done by one man or one institution; it represents the combined efforts of several agencies and heavy expenditures. The Bureau of Standards has not sufficient funds; support must come from the industries. The Bureau of Standards has not enough funds to carry out work now on hand. Indeed there are valuable data buried in the files that should be worked up. **Mr. Hatt.**

REPORT OF COMMITTEE E-7, ON WATERPROOFING.

Committee E-7, on Waterproofing, had intended holding a meeting at the time of this convention, but due to the unexpected illness of the chairman, and the inability to hold a meeting, the following progress report has been prepared for submission to the Institute:

Final appointments on Committee E-7 were only completed on Nov. 17, 1921. A meeting was held in Philadelphia on December 15. With the exception of three members, the entire committee of sixteen attended.

After discussing for some time the scope of the Committee's work, and various other details, the following sub-committees were appointed:

1. Integral methods and plaster coatings.
2. Penetrative surface applications.
3. Bituminous coatings and membranous systems.
4. Non-bituminous surface coatings.
5. Waterproofing concrete.

All sub-committee reports were to be in the hands of the chairman by January 20, so that a definite report for the Committee could be made to this session of the Institute. Various changes however have been necessary, not only in the personnel of the sub-committee, but in the chairmen as well.

Consequently no formal report can be presented at this time. The progress already made indicates that not only will a definite and constructive report be ready for presentation at the next meeting of the Institute, but that there is a real need for just such a committee for the purpose of studying and clarifying some of the misunderstandings which seem now to exist on the subject of waterproofing.

J. H. LIBBERTON, *Secretary.*
ERNEST ASHTON,
MAXMILIAN TOCH (A. H. RHETT),
A. W. STEPHENS,
D. A. ABRAMS,
J. C. PEARSON,
PETER C. SPENCE (R. P. BROWN).

REPORT OF COMMITTEE S-I, ON REINFORCED-CONCRETE CHIMNEYS.

The committee has prepared the substance of a specification for the design and construction of tapering reinforced-concrete chimneys. It is not in final form but is presented this year for discussion.

RECOMMENDED SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF TAPERING REINFORCED-CONCRETE CHIMNEYS.

The intent of these specifications is to provide for the design and construction of tapering reinforced-concrete chimneys and their foundations.

Design.—In designing the chimney the following primary assumptions shall govern:

1. A plain section before bending remains plain after bending.
2. The thickness of the wall at any horizontal section shall be constant.
3. The temperature and torsional stresses shall be properly provided for with reinforcement.
4. The temperature and torsional reinforcement shall be spaced not more than 6 in. apart horizontally, and the distance from the face of the wall to the reinforcement shall not be more than 3 in.
5. A wind pressure due to a wind having a velocity of 100 miles per hour for the entire height of the chimney.
6. Weight of the reinforced concrete—150 lb. per cu. ft.
7. The tensile strength of concrete is to be neglected when computing for tensile stresses in the reinforcing steel.
8. The compression strength of the steel is to be neglected when computing for the compression stresses in the concrete.
9. Maximum allowable compression stress in concrete—350 lb. per sq. in.
10. Maximum allowable tensile stress in open hearth steel—16,000 lb. per sq. in.
11. Maximum allowable tensile stress in cold drawn steel—18,000 lb. per sq. in.
12. Maximum allowable shear on concrete shall not exceed 40 lb. per sq. in., unless shear reinforcement is provided, in which case the shearing strength shall not exceed 100 lb. per sq. in.
13. The full strength of the bars shall be assumed to be developed by the imbedment of the bar equal to forty times its diameter.
14. The foundations shall be reinforced with steel bars to act as a cantilever footing.

Workmanship.—The work shall be done in a first class and workmanlike manner under the supervision of an experienced superintendent or foreman. The chimney shall have a uniform taper from bottom to top on the outside, and shall have at any horizontal section a true outer and inner circle, concentric to the center, and a wall of even thickness throughout the section. All ridges and offsets shall be removed so that the chimney when completed shall have a smooth surface of uniform color.

Forms.—Forms for the shaft shall be of such construction and thickness that they will conform readily to the circular lines of the section but must be well braced and sufficiently strong to prevent bulging. The inner and outer forms shall be set in place so as to insure a concentric alignment and resultant uniform thickness of the chimney wall at all parts of any horizontal section. All joints shall be tight so as to prevent the leakage of mortar. Proper precaution is to be taken to have the forms held in place so that the chimney shaft will at all points be true to the vertical axis.

Materials.—Cement. The cement shall be portland cement which conforms to the requirements of the latest standard specifications of the American Society for Testing Materials. It shall be stored until used, in a building which will protect it properly from the weather.

Fine Aggregate. Fine aggregate shall consist of screened, washed sand, crushed rock, gravel screenings, or air cooled blast furnace slag screenings, graded from fine to coarse, and passing when dry, a screen having four meshes per linear inch. It shall be clean and free from any injurious amount of soft particles, lumps of clay, vegetable loam, organic matter or other impurities.

Coarse Aggregate. The coarse aggregate shall consist of properly graded crushed rock, gravel, or air cooled blast furnace slag. It shall be clean, hard and durable, and free from all deleterious matter.

Water.—Water used for mixing concrete shall be free from oil, acid, alkalies or organic matter.

Steel.—Reinforcing steel shall comply with the requirements of the latest standard specifications for billet steel reinforcement, adopted by the American Society for Testing Materials. Reinforcement shall be of new billet stock, having a tensile strength of from 55,000 to 75,000 lb. per sq. in. Cold drawn steel wire used for temperature and torsional reinforcement shall have a tensile strength of from 75,000 to 85,000 lb. per sq. in. Reinforcement shall be free from scale, paint, coatings of any character, or excessive rust, which would tend to reduce or destroy the bond with the concrete.

Proportions.—The unit of measure shall be the cubic foot. Ninety-four pounds, or one sack of cement, shall be assumed as one cubic foot. Foundation concrete shall be mixed in the proportion of one sack of portland cement to not more than 8 cu. ft. of fine and coarse aggregate, measured separately, and in no case shall the fine aggregate exceed three times the volume of cement. Concrete for the chimney shaft shall be mixed in the proportion of one sack of portland cement to not over 5½ cu. ft. of fine and coarse aggregate, measured separately, and in no case shall the fine aggregate exceed twice the volume of cement. The proportioning of the fine and coarse aggregate shall be such as will produce as dense a concrete as possible.

Machine Mixing.—Concrete shall be mixed in a batch mixer of approved type, which will insure uniform distribution of the materials throughout the mass. After all materials and water are in the drum, the concrete shall be mixed not less than one and one-half minutes, during which time the mixer shall rotate at a peripheral speed of about 200 ft. per min. The volume of the mixed material per batch shall not exceed the manufacturer's rated capacity of the mixer.

Hand Mixing.—Where absolutely necessary to resort to hand mixing it shall be done on a watertight platform. The materials shall be turned at least six times after the water is added and until the batch is homogeneous in appearance and color.

Consistency.—The consistency of the concrete shall be such that it will flow sluggishly in the forms and around the reinforcement. An excess of water shall be avoided because it seriously affects the strength of the concrete and any batch of concrete containing such an excess shall be rejected.

Retempering.—No concrete shall be used that has partly set. Retempering of concrete or mortar which has partially hardened, that is, remixing with or without additional materials or water, will not be permitted.

Placing.—After mixing, the concrete shall be handled as rapidly as practicable into place. Concrete shall be placed in as continuous an operation as possible. It shall be deposited in a manner so as to permit of a thorough compacting and shall be worked in and around the reinforcement by means of spading so as to secure a dense mass, free from voids or fissures. Before placing concrete, the reinforcement is to be placed in correct position. Precautions are to be taken to prevent a subsequent displacement.

The space to be occupied by the concrete is to be free from all debris. When the placing of concrete is to be stopped for such a length of time that the concrete will harden before work is again resumed, the previously placed concrete shall be cleaned from all dirt, scum or laitance. Before any new concrete is placed, the previous work shall be wet and covered with a mortar of one part cement and not over two parts of fine aggregate.

Temperature Below 45 Deg. Fahrenheit.—If at any time during the progress of the work the temperature drops below 45°, the water and the aggregates shall be heated and precautions taken to protect the work from freezing.

Reinforcement.—The vertical rods imbedded in the foundations shall be bent under the foundation reinforcement for anchorage. All splices shall be not less than forty times the diameter of the bars. Splices are to be distributed, and no splices are to be made in the foundation at points at maximum stress.

Ornamental Head.—The top of the shaft shall be built out to form a neat looking crown.

Lining.—The lining is to be built of materials of suitable quality to withstand the temperature and action of the flue gases. The lining is to be of such height and thickness as to properly protect the shaft of the chimney from the action and temperature of the flue gases.

Surface Finish.—All exposed surfaces shall be finished so as to present a smooth surface. Upon completion the shaft shall be washed down with a cement wash.

Cleanout Door.—The chimney shall be equipped with a standard 24x36 in. cast-iron cleanout door and frame.

Pulley and Cable.—The chimney shall be provided with one suitable bronze pulley, anchored to the top of same, and a loop of $\frac{3}{16}$ in. galvanized wire cable extending full height of chimney.

ERIC PLAGWIT, *Chairman.*

DISCUSSION.

Mr. Feldrappe.

M. G. FELDRAPPE.—I cannot agree with the committee in allowing only 300 lb. stress. It seems to me that if we want to promote the construction of concrete chimneys, we ought not to be as conservative as that. Is there any reason for it?

Mr. Plagwit.

ERIC PLAGWIT.—A compressive strength of 350 lb. per sq. in. is the adopted practice of most of the conservative builders in this country. The principal reason for this is that in order to give the chimney sufficient taper, the wall at the bottom would be very small if the stress would run above 350 lb. Furthermore, the cost of the concrete material to go into a concrete chimney is comparatively very small compared with the labor cost, which is the principal item. So whether you put in a few yards more of concrete or not has really no bearing on the cost of the chimney as a whole.

Mr. Feldrappe.

MR. FELDRAPPE.—Why waste it, if it is not necessary? The most conservative codes say 500 lb. for a 1 : 2 : 4 mix.

Mr. Plagwit.

MR. PLAGWIT.—We fear we cannot be too conservative in designing chimneys, because chimneys are subject to stresses which other things are not subject to, such as temperature and torsion stresses caused by the wind blowing against the chimney. The chimney does not sway back and forth like a pendulum, but in such a way that the center of the top section describes an ellipse with the long axis parallel to the end. Then again, a wind may blow in one direction for half an hour and then turn clear around and blow in the other direction; that is another reason for the compressive stress.

Mr. Feldrappe.

MR. FELDRAPPE.—When you design a chimney, you design it to take care of these stresses, don't you?

Mr. Plagwit.

MR. PLAGWIT.—Yes, we do, but nevertheless the stresses are one of the reasons why we put the compression stress as low as 350.

Mr. Lowell.

JOHN W. LOWELL.—Committee S-1, on Concrete Chimneys, of which I am a member, realizes that the data at its disposal is sufficient to enable it to draft a specification covering only the general features of design and construction of concrete chimneys.

No doubt temperature reinforcement of chimneys should be given greater consideration in our specification, but before this can logically be considered by the Committee, we must have before us not only theories, but those theories which have been checked by experiment or actual practice. To get the subject of temperature stresses before the members of the Institute and other engineers who may wish to study this subject, I, as an individual, should like to present the following discussion of circumferential temperature stresses.

That internal forces are set up in the chimney shell due to heat is undisputed. It is likewise readily seen that these forces are functions

Mr. Lowell. of the thickness of the shell and diameter of the chimney as well as the drop in temperature between the inside and outside of the wall.

Still, after building concrete chimneys for about 20 years, designers and builders of chimneys, so far as I have been able to learn, do not use even an empirical formula to determine how these forces can be taken care of to lessen the damage to chimneys. Cracks occur in most all chimneys, in some much worse than in others. Yet, it is evident that this does not arouse our curiosity when we stop to realize that practically no change has been made during the last 15 years in the practice of reinforcing to take care of temperature stresses.

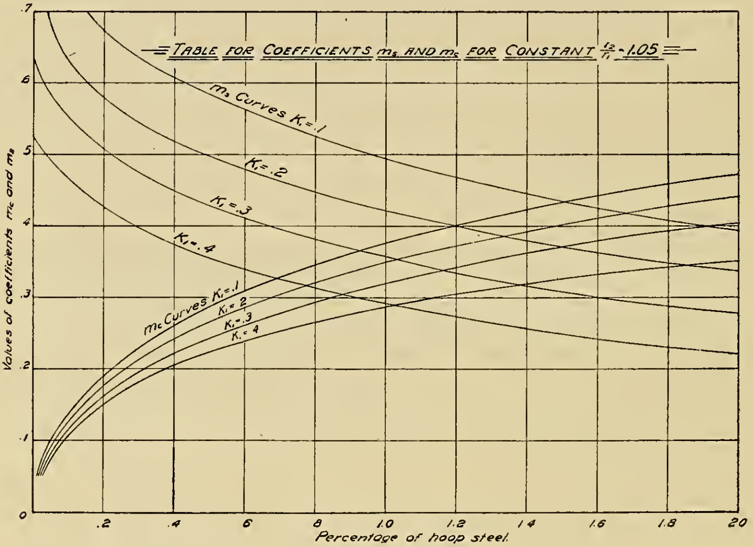

FIG. 1.

From the nature of internal and external conditions, the derivation of a theoretical formula should not be difficult. The testing of stresses and strains in new chimneys being heated for the first time would make a very interesting problem. Also, tests of annular sections heated under control would likewise produce valuable data.

This problem is of sufficient importance to be brought to the attention of the various engineering experimental stations and to the engineering colleges as a thesis subject. It is hoped that the results of such study and experimentation that may be made will be brought to the attention of the Institute's chimney committee.

A study of the various text books and references relating to concrete chimneys showed that there is no theory or other data developed sufficiently to be usable in designing various sized chimneys. However, Turneaure & Maurer have built up a theory on the assumption of a constant modulus of elasticity for concrete in compression within the temperature range and a straight line temperature gradient through the shell.

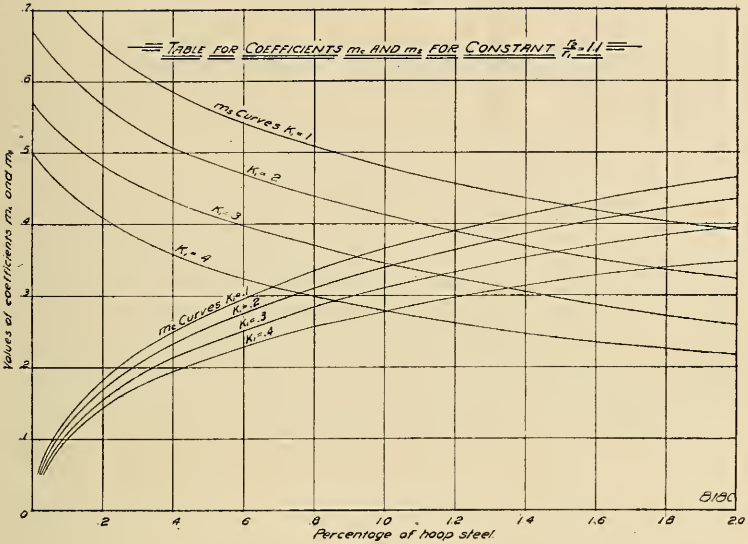
TABLE FOR COEFFICIENTS m_c AND m_s FOR CONSTANT $\frac{r_2}{r_1}=1.1$
m_s Curves $K_1=.1$
$K_1=.2$
$K_1=.3$
$K_1=.4$
m_c Curves $K_1=.1$
$K_1=.2$
$K_1=.3$
$K_1=.4$
Values of coefficients m_c and m_s
.7
.6
.5
.4
.3
.2
.1
0
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
Percentage of hoop steel
818C

FIG. 2.

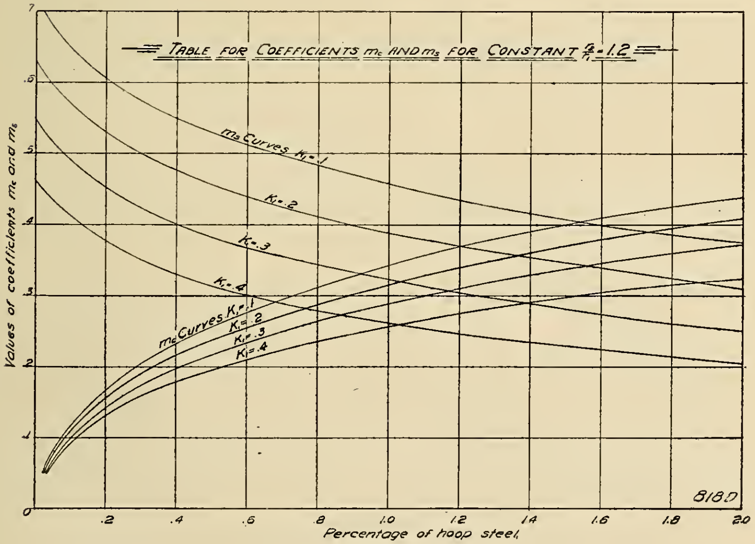
TABLE FOR COEFFICIENTS m_c AND m_s FOR CONSTANT $\frac{r_2}{r_1}=1.2$
m_s Curves $K_1=.1$
$K_1=.2$
$K_1=.3$
$K_1=.4$
m_c Curves $K_1=.1$
$K_1=.2$
$K_1=.3$
$K_1=.4$
Values of coefficients m_c and m_s
.7
.6
.5
.4
.3
.2
.1
0
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
Percentage of hoop steel
818D

FIG. 3.

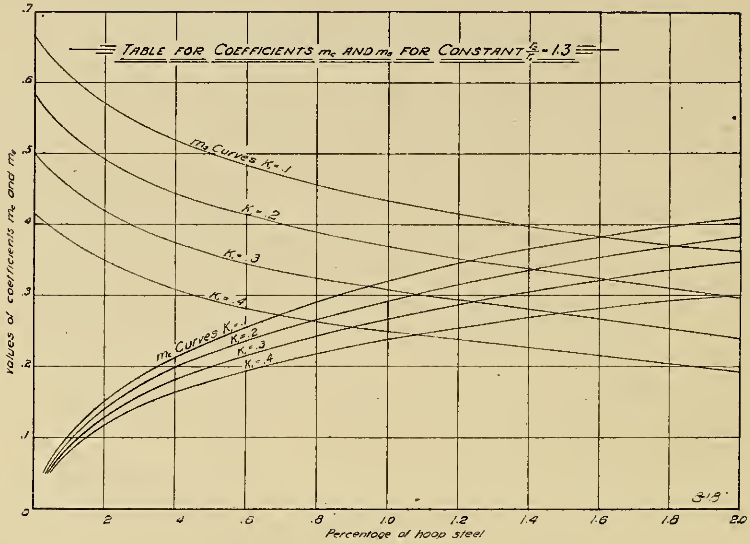
TABLE FOR COEFFICIENTS m_c AND m_s FOR CONSTANT $\frac{f_2}{f_1} = 1.3$
m_s Curves $K_1 = .1$
$K_1 = .2$
$K_1 = .3$
$K_1 = .4$
m_c Curves $K_1 = .1$
$K_1 = .2$
$K_1 = .3$
$K_1 = .4$
Values of coefficients m_c and m_s
.7
.6
.5
.4
.3
.2
.1
0
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
Percentage of hoop steel
818

FIG. 4.

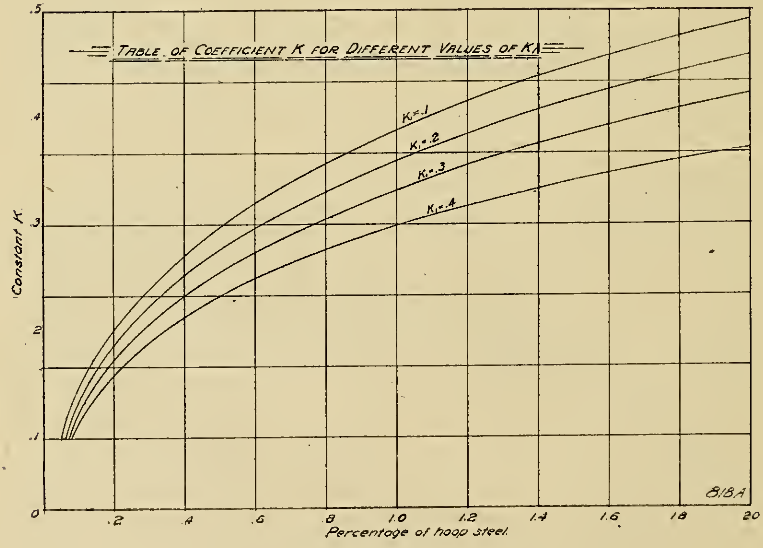
TABLE OF COEFFICIENT K FOR DIFFERENT VALUES OF K_1
$K_1 = .1$
$K_1 = .2$
$K_1 = .3$
$K_1 = .4$
Constant K
.5
.4
.3
.2
.1
0
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
Percentage of hoop steel
818A

FIG. 5.

Mr. Lowell.

This theory is given in "Principles of Reinforced-Concrete Construction," by Turneaure & Maurer, p. 413.

To make this theory usable and to bring it to the more direct attention of those who may wish to study the problem of temperature stresses, I have developed, and present to the committee as a part of this discussion, charts which cover a wide range of chimney sizes met with in design.

It is not my intention to suggest that cracks should be eliminated, but I do think they should be controlled in size and distribution. I believe this can be accomplished readily if the tensile stress in the steel reinforcement be kept within two-thirds of its elastic limit and a cold drawn wire mesh is used with wires spaced about 4 in. and not greater than 6 in. apart horizontally, and if the compressive stress of the concrete is kept within two-thirds of its rated ultimate strength.

The following formulas (1) and (2), to be used with the charts presented hereafter in calculating the circumferential temperature stresses and area of reinforcement, are taken from "Principles of Reinforced-Concrete Construction," by Turneaure & Maurer, p. 413-419.

$$(1) \qquad f_c = \pi K E_c m_c \qquad\qquad (2) \qquad f_s = \pi K E_s m_s$$

$$m_c = \left[1 - (1-k)\frac{t}{r_1}\right] k \qquad\qquad m_s = \left[\frac{(1-k)\left(1+\frac{kt}{r_1}\right)}{\left(\frac{r_2}{r_1}\right) - \frac{k_1 t}{r_1}} - k_1\right]$$

Values of m_c and m_s for various values of p and k_1 are shown in Figs. 1 to 4 where $n = 15$.

Values for k and p for various values of k_1 where $n = 15$ are shown on Fig. 5. The curves of Fig. 5 are plotted from the formula

$$\frac{k - \frac{1}{2}k^2 + npk_1}{(1-k)\left(k + \frac{r_1}{t}\right)} = log\left(1 + \frac{kt}{r_1}\right) + \frac{np}{\frac{r_2}{t} - k_1}$$

k_1 = distance of circumferential steel reinforcement from outer surface of concrete shell divided by total thickness of shell;

k = ratio of depth of neutral axis of a section from the compressive face;

t = thickness of concrete shell at the section under consideration;

r_1 = inner radius of shell;

r_2 = outer radius of shell;

π = difference of temperatures of concrete at outer and inner faces;

K = coefficient of expansion of concrete and steel;

E_c = modulus of elasticity of concrete in compression;

E_s = modulus of steel;

f_c = temperature unit stress in concrete (circumferential) at inner face; and

f_s = temperature unit stress in circumferential steel.

[A written discussion of this report by Edward Godfrey appears on p. 315 of this volume.]

REPORT OF COMMITTEE S-2, ON REINFORCED-CONCRETE HIGHWAY BRIDGES AND CULVERTS

Waterproofing, Expansion and Construction Joints and Drainage.—These considerations are of such importance in reinforced-concrete bridge design that your committee felt the necessity of giving preference to them over other subjects for which they have planned and are gathering data to report upon.

It has been found that these considerations have very often been given perfunctory thought and many structures show the effect of this neglect. The embodiment of good practice is not confined to theoretical economies based upon actual quantities of concrete, steel reinforcement and other materials; of equal or greater importance is the surety of a positive water repellent protection, the development of an effective expansion joint, the determination and arrangement of construction joints that will not weaken the structure nor exceed the capacity of a plant layout commensurate with the magnitude of the work and finally the distribution of a drainage system to down-spouts and so arranged as to minimize the concentration of water at the expansion and construction joints or at the termination of the flashings of the waterproofing medium. Thus it is seen that these four considerations are correlated factors so closely allied to each other and of such importance that the details of all must be investigated and developed coherently, not as an after thought, but bearing materially on the type of the superstructure selected.

Waterproofing.—In general concrete work, waterproofing systems can be divided into three classes, the membrane, the integral and the painting or plaster treatment.

The membrane system consists of the laying or building up of a water repellent blanket over the surface of the concrete—the bridge floor, the back of a barrel arch, the inner face of a parapet wall or any other surface exposed to drainage or to hydrostatic pressure. The waterproofing medium is a bituminous product, either an asphalt or a coal tar pitch. The bitumen, applied hot to the surface of the concrete, is reinforced and held in place by laying a treated or untreated sheet of felt, paper, burlap, cotton cloth or a combination of these materials after each swabbing of bitumen. The number of plies vary from two to five depending upon the kind of membrane used.

The integral method consists of the incorporation of a compound, either in paste, liquid or powdered form, with the concrete ingredients during the mixing to give greater flowbility by lubrication of the surfaces of the aggregate or resulting in further hydration of the cement, both principles intended to produce maximum density or such other chemical action intended to decrease the permeability of the concrete.

The painting or plaster treatments consist of the application of bituminous or other paints, or the integral method used in cement mortar and applied like plaster to the surface of the concrete. A rubbing treatment also has been used in which the compound is worked into the surface of the concrete intended, by act of chemical or physical assimilation, to eliminate permeability of the concrete.

It is not considered that the integral methods have a function in waterproofing concrete bridges of such size that construction joints are unavoidable. The interruptions in the continuity of the surface that is to be protected by reason of both the incipient cracks that do develop regardless of the amount of reinforcement and the necessary expansion and construction joints make such methods inapplicable.

The aggregate may be so proportioned, treated and mixed that the resulting mixture would give a concrete of the highest degree of density, but the fact must not be overlooked, in the final analysis, that the actual density of the finished product depends largely in the manner in which the concrete is deposited in the forms. This is not controlled by the perfection of the mix, physically or chemically, but there remains a personal equation which is not fully controlled on any work.

The membrane system of waterproofing is a guard against possible defects in concrete bridge construction and has been used in a most effective manner. Regardless of the surface porosity or the incipient cracks that may develop, the expansion and construction joints necessary to the work, the membrane system is real insurance against disintegrations known to unprotected structures at these points of susceptive weaknesses.

At present this limits the field to the bituminous compositions, asphalt and coal tar pitch. As between coal tar pitch and a properly refined asphalt your committee is of the opinion that the physical properties of the latter is better suited to meet the outstanding requirements of service; because of the close proximity of the material to the surface of the structure, it is subjected to structural movements and shock of impact through wide range of temperature. Coal tar pitch is more susceptible to changes in temperature than asphalt; its consistency changes from an extremely brittle solid at 40 degrees F. to a very soft fluid at 115 degrees. At the lower temperature the quality of ductility is completely lacking.

Not all asphalts, however, are suitable for waterproofing; some of them, in fact, being very poor waterproofing agents by reason of the fact that they lack the essential physical properties, or contain impurities prejudicial to their life.

The American Society for Testing Materials and the American Society of Civil Engineers have established standard tests for bituminous materials and have described in minute detail how such tests shall be made. From these tests it is entirely possible to predetermine with considerable accuracy how bituminous materials will behave in service and the relative life that may reasonably be anticipated from them. Therefore, reliance should not be placed on trade contentions of a material of inexplicable composition. Through lack of proper investigation the profession has been badly misled. Specifications can and, in the opinion of your committee, should be written which describe by standard tests previously mentioned the quality of the material desired just as the quality of the aggregates, cement, steel and other materials is described.

In recommending the following tentative standard the committee is not acting entirely upon its own knowledge of the subject, but has consulted with independent authorities on bituminous products.

TENTATIVE STANDARD FOR BITUMINOUS WATERPROOFING

(1). The asphalt used should have the highest possible percentage of bitumen, as the bitumen content in an asphalt is the waterproofing content of value. Many asphalts will test 99½ per cent pure—98 per cent would not be an unreasonable requirement.

(2). The asphalt should be relatively soft at 32 deg. F. and relatively stiff at 115 deg. F. In other words the ratio between the consistencies of the material at these temperatures as measured by the standard penetration tests, or the "susceptibility ratio," should not be greater than 9 with a maximum penetration (standard test) at 115 deg. F. of one centimeter.

(3). The ductility at 77 deg. F. should be not less than 15 cm. and should be as great as possible. There are several reasons for this. An asphalt with a low "susceptibility ratio" can be produced, as has been ascertained, by the oxidizing process only. This process must be conducted with great care since, if an asphalt is over oxidized it may for a short period after being manufactured, appear to exhibit the required properties, but be subject thereafter to a rapid disintegration which changes it into a "cheesey" substance entirely devoid of waterproofing value. It is not believed possible to over oxidize an asphalt and still retain the minimum ductility mentioned. Adhesion to the concrete and to the membrane is undoubtedly of great importance; since the ductility is perhaps the best gage of the adhesive value of the material, it should be as high as possible consistent with the penetration requirements. This statement is applicable regardless of temperature, but it is even more important at low temperature than at high; a ductility of at least 3 cm. at 40 deg. F. is desirable.

(4). The losses in weight and in penetration through heating for five hours at 325 deg. F. should be a minimum—not more than 1 per cent and 25 per cent, respectively. This test is an accelerated aging test and is intended to insure the durability of an asphalt. The desirability, therefore, of a material undergoing a minimum change when subjecteded to it is manifest.

From a waterproofing standpoint these tests, it is believed, comprise all the important tests prescribed by the two organizations mentioned, but it is to be noted that they have a significance only when construed together,—that no correct conclusion can be drawn from any one or from several of them unless all are included.

Bidders should be required to submit samples of the material they propose to furnish, with a statement from a reliable testing laboratory that they comply with the above requirements; the materials delivered to the job should be tested at least two weeks prior to their use. Samples should be taken from each car load of materials delivered on the work, and tested in similar manner.

The function of the membranc is to hold the bitumen in place and to reinforce the blanket. It seems desirable to thoroughly impregnate the membrane with the waterproofing asphalt to preclude a possible chemical action with unlike materials. Tests of all membranes show greater strength after treatment than before. There is a possibility of reduction in strength, if membrane is burned in process of saturation. Elasticity, flexibility, tensile strength, durability and bond strength with the asphalt are essential characteristics to be considered in selection of the membrane reinforcement.

Waterproofing Details of Design.—Plans should show full detail of the waterproofing system, including provisions for flashing ends of the membrane, for folding membrane at expansion joints to guard against fracture due to excessive movement and for flashing at the down-spouts. Plans should avoid the fastening of street car rails with anchor bolts to the floor in an endeavor to eliminate the tie since much trouble has resulted by reason of this detail, which arrangement prohibits a positive continuous waterproofing blanket. If possible, the waterproofing membrane should not be terminated on face of curb in the gutter, as this places the termination at the point of almost perpetual dampness. Several successful schemes covering usual conditions of design are illustrated in Figs. 1 and 2.

Preparation of Surface.—All concrete surfaces to be waterproofed should be carefully floated to a smooth surface, embedding or removing such large projecting aggregate that might puncture the membrane.

Clean surface thoroughly by use of a stiff corn broom followed by a soft broom for removal of dust and finer particles. It is desirable to have the surface perfectly dry, but a little dampness or even a slight greenness of concrete will be allowable when the saving of time is an important element.

Protection of Membrane.—The waterproofing membrane should be protected, in the case of a barrel arch supporting earth fill, by a 1½ in. layer of 1 : 3 cement-sand mortar or by a layer of common brick laid with open joints, filled with cement or asphalt or by any other equivalent protection. This protection should be applied immediately after the waterproofing has been laid to prevent it from being punctured by subsequent construction operation. An asphalt pavement never should be applied directly over a waterproofing blanket, as subsequent repairs to pavement could not be made without puncturing the membrane; here is another function of the protection coat. Vertical wall work in contact with back fill is protected by one-half inch plaster coating which can be made to adhere readily to the asphaltic surface.

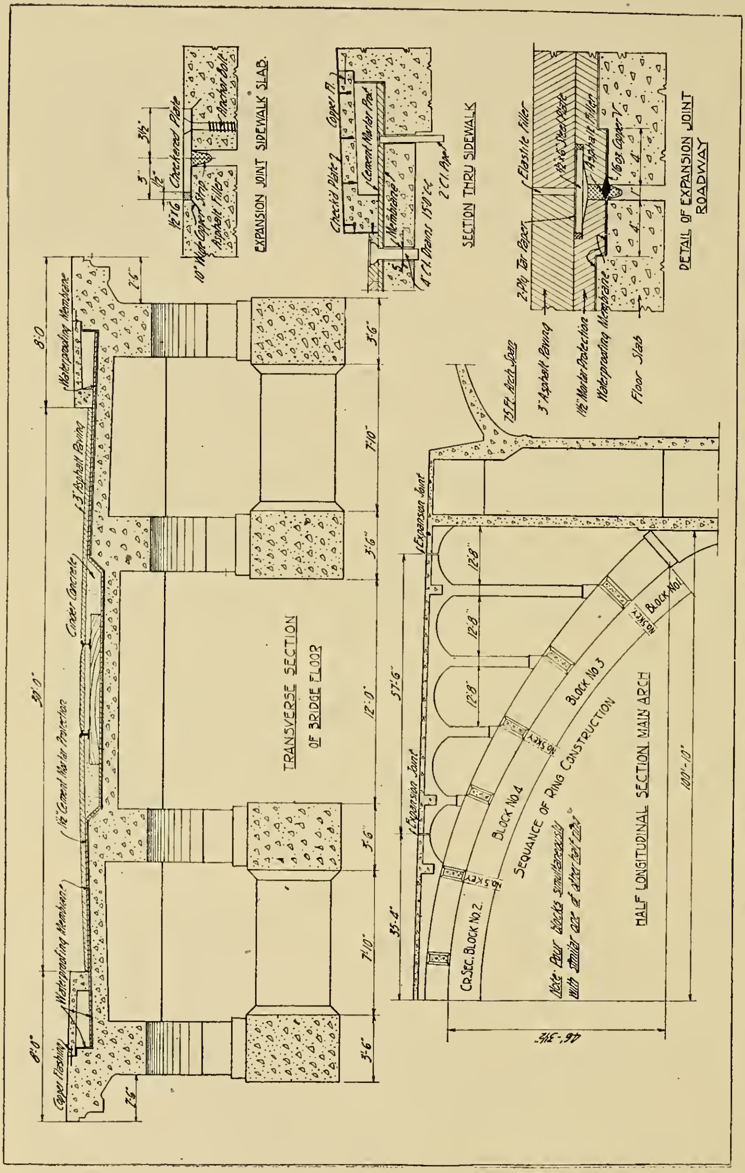

FIG. 1.—DETAILS OF A TYPICAL REINFORCED-CONCRETE ARCH BRIDGE.

Expansion Joints.—Since the coefficients of expansion of steel and concrete respectively are approximately equal, reinforced-concrete structures of appreciable size require expansion joints in manner similar to practice controlling steel design. By reason of the tensile strength of built up sections of a steel member, structural steel carries the stress due to temperature changes over greater lengths without rupture, than the less elastic reinforced-concrete section. Therefore, it is necessary to place expansion joints in concrete construction at even closer intervals than would be necessary in steel structure of similar magnitude.

In concrete structures the distance apart of these joints depends upon the type of structures and no specific recommendations relative thereto can safely be given. Recommendations covering general considerations can be made, however, that will give good index to conclusions for the varying conditions encountered in concrete design.

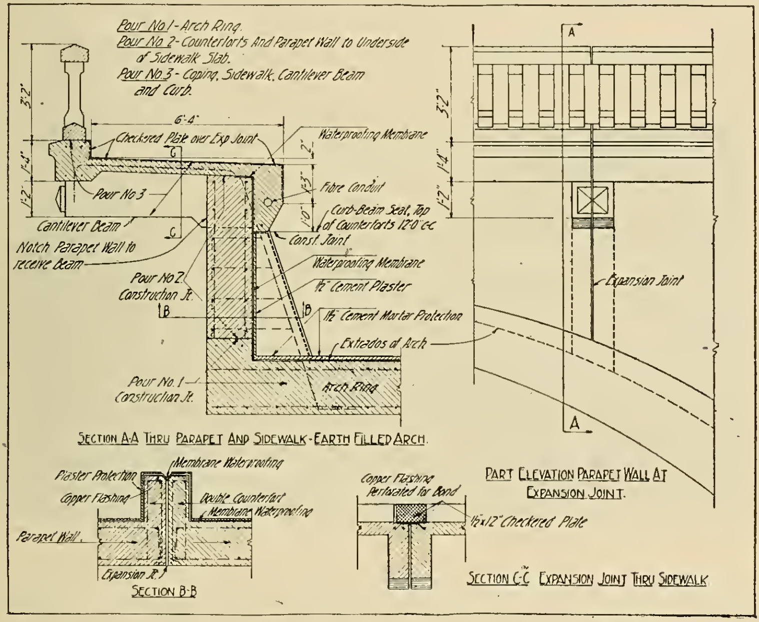
Pour No 1 - Arch Ring.
Pour No 2 - Counterforts And Parapet Wall to Underside of Sidewalk Slab.
Pour No 3 - Coping, Sidewalk, Cantilever Beam and Curb.
3'-2"
1'-4"
1'-2"
6'-4"
Checkered Plate over Exp Joint
Waterproofing Membrane
2"
1'-3"
1'-0"
Fibre Conduit
Curb-Beam Seat, Top of Counterforts 12'-0" c-c
Const. Joint
C
C
Pour No 3
Cantilever Beam
Notch Parapet Wall to receive Beam
Pour No 2 Construction Jt.
B
B
Waterproofing Membrane
½" Cement Plaster
1½" Cement Mortar Protection
Extrados of Arch
Pour No. 1 Construction Jt.
Arch Ring
SECTION A-A THRU PARAPET AND SIDEWALK - EARTH FILLED ARCH.
A
A
Expansion Joint
PART ELEVATION PARAPET WALL AT EXPANSION JOINT.
Membrane Waterproofing
Plaster Protection
Copper Flashing
Double Counterfort
Membrane Waterproofing
Parapet Wall
Expansion Jt.
SECTION B-B
Copper Flashing Perforated for Bond
½x12" Checkered Plate
SECTION C-C EXPANSION JOINT THRU SIDEWALK

FIG. 2.—DETAILS OF EXPANSION JOINTS THROUGH SIDEWALK.

Broadly speaking, there are three types of highway bridge design requiring expansion joints; first, the arch type with superstructure composed of transverse or longitudinal walls supporting a floor system or with back fill of earth where the sidewalk construction is cantilevered from a parapet wall; second, the T-beam or rectangular slab design supported on columns or bents; third, the multiple way reinforced flat slab structure on columns. The latter type has not been extensively used but there is a wide field for its application.

In the development of expansion joint details the following considerations should be borne in mind. In the arch type with superstructure, expansion joints in the superstructure must provide for movement in both a vertical and horizontal direction—up and down for rise and fall of the arch ring and in a horizontal direction due to elemental changes of length in the various members of the floor system. Where transition for these non-concurrent movements from one section to another is attempted in the superstructure by sliding or vertical surfaces in contact, there is the imminent danger of a shearing failure in these sections in contact caused by excess of frictional or tortional action over the strength of the section in shear. The cantilever joint shown in Fig. 1 has been found to circumvent such a failure. The floor beams and slab of a particular span of superstructure are cantilevered from columns or bents to form a clear cut opening at the center of the span, providing free action in either horizontal or vertical direction. Joints placed at the quarter points of the span, approximately at the point of contraflexure in the arch ring, and at the pilasters adjacent to the piers have proved successful for spans of 90 to 200 ft. This arrangement divides the floor system over each arch in three sections and eliminates an extremely large moment in the floor system over the haunch of the arch due to a maximum vertical movement at crown of arch. The amount of moment or tension in the floor system due to vertical movement of the main arch ring varies with the ratio of rise to span; the flat segmental arch causes greater moment than the semicircular arch of equal span. The principle or effect of the cantilever joint in floor system can be secured in the parapet wall construction of a barrel arch supporting earth fill, by vertical joint through the walls and side-

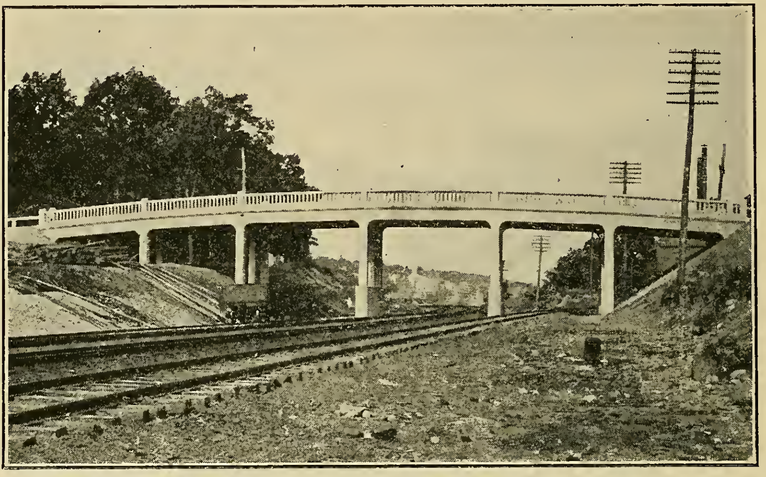

FIG. 3.—AN EXAMPLE OF A LIGHT GIRDER BRIDGE OF CONCRETE.

walk as shown in Fig. 2; here two separate abuting counterforts and cantilever beams take end reactions.

For the second type, the T-beam or rectangular slab design supported on columns or bents, the action to be controlled is in the horizontal direction only. In the T-beam type there arises the difficulty of transferring the movement due to temperature changes from the larger through the smaller members as from the deep beams through the slabs. Regardless of the length of structure the slab should be substantially reinforced in transverse direction to distribute stress or prevent a cumulative strain at change of sections. The usual T-beam type as exemplified by Fig. 3 has been built in total length of 179 ft.; this structure has four 25 ft. and two 30 ft. 10 in. spans without an expansion joint.

The ratio of span length to height of columns or bents should be taken into consideration. Fig. 4 shows T-beam bridge of six 50 ft. spans and two 40 ft. spans; maximum height of the structure is 50 ft. To incorporate a sliding joint for spans of this length would require heavy bents or piers. In this case a double row of columns was used to take the end reactions, cutting the structure in three separate and distinct divisions. The stress resulting from the movement of 150 ft. section is taken by bending or slight flexure of the columns. Each beam is supported directly on column, thus eliminating a heavy beam cap over the columns. Fig. 5 shows failure of tar paper lined expansion joint where irregular construction caused frictional resistance to exceed shearing strength of the pier cap.

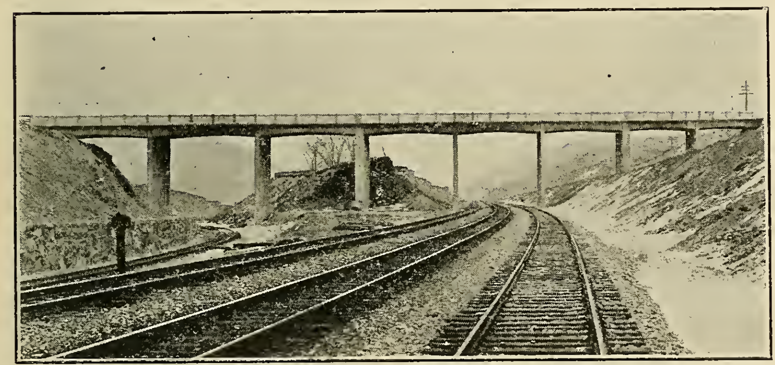

FIG. 4.—MILWAUKEE ROAD BRIDGE, LACKAWANNA R. R., NEAR CLARK'S SUMMIT, PA.

For sliding joints, plates of steel, copper, zinc or lead and multiple layers of fabrics or combination of these different plates have been used to reduce friction resistance. Where the bearing covers a wide area there is the danger, if heavy plates are not used, that the sliding medium or plate will conform to a possible uneven surface of a bridge seat and cause a crack similar to one shown in Fig. 5. Therefore, great care must be exercised in preparation of the sliding joint, or bridge seat, as it is generally called.

The third type, the flat-slab construction, can be built in greater lengths between expansion joints than either of the preceding types. Because of the uniform cross section everywhere of both concrete and steel, the slab offers better resistance to temperature changes, the movement being distributed more uniformly throughout its entire length. Expansion joints in this construction are formed by double row of columns, and have been placed 418 ft. apart in structure over railroad tracks where close lateral clearances precluded double row of columns. A length not exceeding 250 ft. would be better practice.

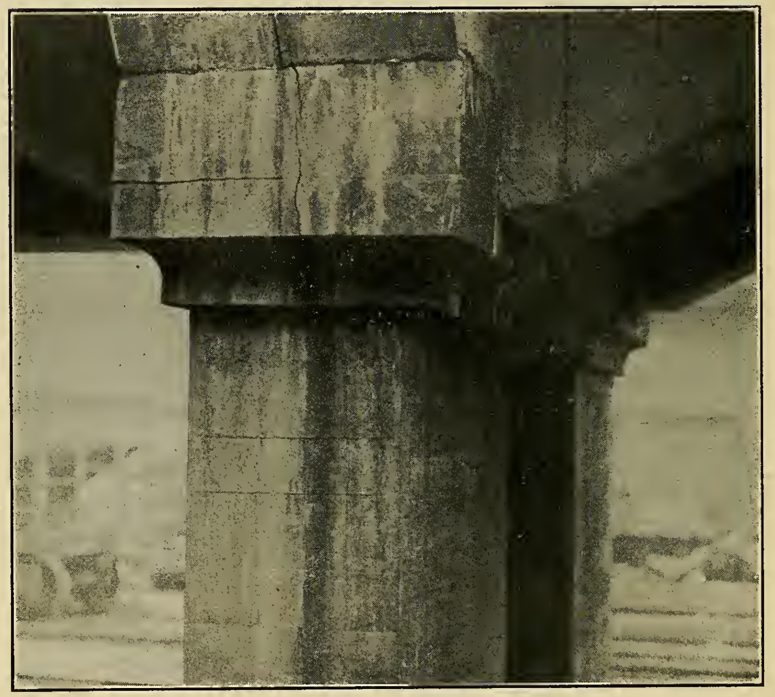

FIG. 5.—CRACKS THROUGH BRIDGE PEDESTAL.

Construction Joints.—Plans should show distinctly the location of possible construction joints. This detail is a decided help to the contractor, enabling a more intelligent analysis of the construction problems and avoids confusion and misunderstanding during the progress of the work. The designer, in arranging the location of these joints, should bear in mind and be able to judge the capacity of plant required commensurate with the magnitude of the work. For example, if a one-yard mixer is sufficient, the average volume of concrete for an 8 hour operation would be approximately 160 cu. yd., an average of a batch every three minutes. It is possible to exceed this by 40 cu. yd., but operation is without interruption. Therefore, any larger section poured in continuous operation would require overtime or double shift.

The lengths of reinforcing rods should be predetermined so that no excessive length of dangling rods project beyond the construction joint to make the placing of the concrete a very difficult operation. The projecting lengths beyond the bulkhead forming construction joint should be long enough to develop bond strength with the next lift of bars. In beam or slab construction, the joint is placed at point of least shear or maximum moment. No lap of bars should be made at this point unless absolutely necessary; then in addition to the usual lap to develop bond strength the bars should be clamped together by mechanical means.

Construction joints for gravity wall work should not be placed at greater intervals than 30 ft. By addition of temperature steel in reinforced walls, they can be poured with safety in sections 50 ft. in length. Gravity walls should be built preferably in one operation from footing course to the underside of the coping, and this may shorten the length of the section in order that the capacity of the plant is not exceeded. Vertical joints in wall construction are far more desirable than the horizontal joint which usually develops into an unsightly crack by reason of the formation of an efflorescence and gradual surface disintegration in spite of extreme precautions in removal of sludge and cleaning the joint before the subsequent operation. It is good practice to dovetail adjoining vertical sections by form insert and to further seal the joint against percolation of drainage by painting first exposed surface with heavy coat of asphalt. If back fill carries considerable water embed copper flashing in the joint in addition to the above precautions. Before placing the copper strip punch ends to be embedded with holes 2 to 3 in. in diameter to insure good bond. The center of the copper strip should be crimped with 1-in. "V" to allow for movement at the joint.

Avoid placing construction joints at sidewalk level or along plane or extrados of a barrel arch or any other plane where water is liable to be concentrated. In Fig. 1, the coping was carried above the sidewalk level simultaneously with sidewalk construction without measurably increasing the cost of construction. The same figure shows a curb section built with the arch ring. In event of the failure of the waterproofing blanket there is no concentration of water at the construction joint. The figure also shows other construction joints and how the waterproofing membrane was positively flashed and lines of termination sealed by application of waterproofing preceding the pouring of sidewalk and curb. These are problems of drainage, waterproofing and construction joints that show correlation of consideration.

In long span arch rib construction, the arch can be divided into voussoirs or blocks and keys as shown in Fig. 1, the sections being poured in numerical order indicated. This arrangement precludes continuous work over 8 hours for an average mixing plant that would probably meet the conditions of construction of the remaining part of the structure; the arrangement has another value in the uniform loading of the centering and the maximum development of shrinkage of the concrete before the keys are poured. Barrel arches which are limited in span as compared with the rib arch, usually not exceeding 100 ft. span, can be poured continuously from skewbacks carrying the work simultaneously from both ends to the crown.

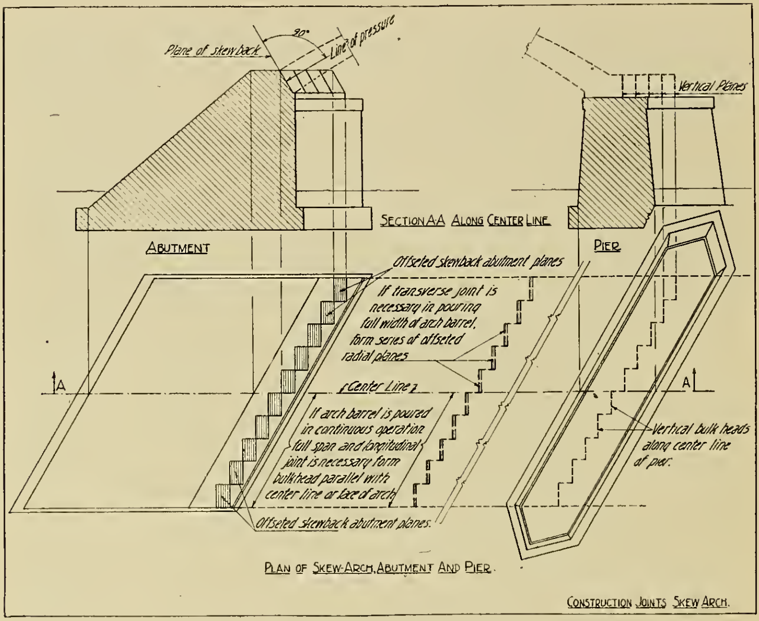
Plane of skewback
30°
Line of pressure
Vertical Planes
Section AA Along Center Line
Abutment
Pier
Offseted skewback abutment planes
If transverse joint is necessary in pouring full width of arch barrel, form series of offseted radial planes
A
Center Line
A
If arch barrel is poured in continuous operation full span and longitudinal joint is necessary form bulkhead parallel with center line or face of arch
Vertical bulk heads along center line of pier.
Offseted skewback abutment planes.
Plan of Skew-Arch, Abutment and Pier.
Construction Joints Skew Arch.

FIG. 6.—DETAILS OF CONSTRUCTION JOINTS IN A SKEW ARCH.

The action of a skew arch is a much mooted question. The safe design of a skew arch depends largely on the manner in which construction joints are fixed. Skew arches are designed on the skew span measured along the longitudinal axis of the bridge. The piers and the abutments should be proportioned in the same manner. Therefore, the arches should be built in laminae parallel with the face or center line of the bridge. The skewbacks of the abutments and piers should be a series of offseted planes set radially with skew axis to take normal thrust of the arch. (See Fig. 6.) If construction joints are necessary in skew arch barrel in transverse direction they should be fixed in manner similar to skewback of the abutment. The main arch reinforcement should be placed parallel with longitudinal axis of the arch. Following these recommendations a highway arch has been built on compact earth foundation on skew of 45 deg., span 83 ft. 3 in., rise 16 ft., and width, face to face of parapet walls, 22 ft.

The committee will have for consideration in formulating report for the next convention, a thorough study and investigation of the action of skew arches. Preliminary information concerning this work indicates that the arch thrust is not nearly uniform along the abutment, being very much heavier near the obtuse angle of the bridge. In some cases it was found to be negative at the acute angle. Then to avoid this eccentric pressure it would seem imperative to eliminate absolutely any transverse joints and to specify construction of the arch ring in comparatively narrow laminae as measured by the length of span. In this manner there could be no transfer or concentration of load to element of the shortest span length.

Drainage.—The outstanding consideration in detail of drainage of a bridge floor is to lead the water to down-spouts placed at close intervals so that no large amount of water is carried over long distances or no large amount of water led to a single down-spout which would tend to choke up by ice formation in freezing weather. If possible, dykes should be built integrally with the floor slab adjacent to the expansion joint, and the floor ramped away therefrom by grade to down-spouts placed apart from the joint. An objection often given to expansion joints is the difficulty in making them watertight. By effective placement of copper flashing, by carrying membrane continuously over the joint and by further protection of plates, as shown in Fig. 1, the expansion joint can be made absolutely watertight. In sidewalk and coping construction above the floor, copper flashing has been used effectively in the construction joints not fully protected by the membrane seepage followed by slight unsightly surface disintegration.

Avoid placing down-spouts in concrete sections without due regard for the expansion of a metal drain pipe; this consideration has been satisfied by wrapping pipe with several plies of tar paper.

Avoid placing outlet of drain where the drip will pour over a concrete surface or overhead where icicle formation will endanger life. It has been found that formation of icicles can be prevented to a large extent by placing perforated copper baskets filled with rock salt at the top of each drain pipe.

In all retaining wall construction place weep holes at ground level to drain back fill. In case of the back fill carrying unusual amount of water, trench drains have been built up around the weep holes to facilitate free flow of drainage. Special attention should be given to the drainage of U shaped abutments. There is a tendency in determining the size of weep hole or drains to fix the size by actual amount of water to be carried. These should be made exceptionally large to avoid being clogged by dirt, and if so, that they might readily be cleaned.

Conclusion.—The importance of these details cannot be over estimated. The technical press has recently reported several complete failures of surface disintegrations, due, no doubt, in some measure, to inadequate provision in design of these all important considerations. Concrete offers many advantages over structural steel in bridge design, and these advantages can be lost or enlarged upon, depending on the amount of progress made in correlating problem of waterproofing, expansion, and construction joints and drainage.

J. H. Ames
G. F. Burch
R. L. Cochran
A. B. Fletcher
G. A. Hool
H. M. Horn
C. B. McCullough
C. T. Morris
J. W. Mueller
W. J. Titus
C. M. Upham
C. G. Wickline
O. L. Grover
E. F. Kelley
A. C. Irwin, *Secretary*
A. B. Cohen, *Chairman*

Committee S-2.

REPORT OF COMMITTEE S-5, ON REINFORCED-CONCRETE HOUSES.

The report of the Committee consists of the following "Recommended Practice for Concrete House Construction."

EMILE G. PERROT, *Chairman.*

RECOMMENDED PRACTICE FOR CONCRETE HOUSE CONSTRUCTION

Section 1. This recommended practice shall apply to the construction of houses not over three stories in height and not exceeding 32 ft. in height between top of first floor and under side of third floor ceiling.

BASEMENT AND FOUNDATION WALLS

Section 2. (*a*) *Monolithic Walls*: Basement and foundation walls of monolithic concrete shall be not less than 8 in. in thickness and shall be supported on a concrete footing or basement floor sufficient to prevent settlement of the building. The monolithic character of a concrete wall may be assumed as effective in distributing concentrated loads in determining the required bearing area on the soil. In analyzing the ability of basement walls to resist the horizontal pressure of the earth, the weight of the superstructure shall be considered as a vertical force against over-turning.

(*b*) *Precast Units:* Basement walls of precast units equal in length to the height of basement and anchored at the bottom to footing, or basement floor, and at the top to first floor, construction shall be not less than 8 in. thick, if of plain concrete, and if reinforced concrete they shall be designed to carry the vertical loads plus a horizontal earth pressure equivalent to the hydrostatic pressure of a liquid weighing 50 lb. per cu. ft. These units shall conform in quality of concrete and absorption requirements to the Specifications and Building Regulations of the American Concrete Institute now in force.

(*c*) *Concrete Block, Brick or Tile:* The thickness of basement walls of concrete hollow block, brick or tile shall be not less than 8 in. and shall be at least 1 in. thicker than the thickness of the outside wall of first story of superstructure. Concrete block or tile used for basement walls shall conform to the requirements of the report of Committee P-1 for 1922, American Concrete Institute.

(*d*) *Filler Walls Between Columns or Piers:* Concrete slabs used as filler walls between columns or piers shall be designed for the moment produced by earth pressure assumed to be horizontal and amounting to the hydrostatic pressure of a liquid weighing 50 lb. per cu. ft.

WALLS OF SUPERSTRUCTURE

Section 3. (*a*) *Monolithic Walls:* Single, solid exterior bearing walls of plain concrete shall be not less than 6 in. thick. Solid walls having 0.2 per cent or less reinforcement shall be considered as plain concrete and walls containing reinforcing steel in excess of 0.2 per cent shall be considered as reinforced concrete and shall be designed for a uniform wind load of 25 lb. per sq. ft. on the vertical projection of exposed surface. The thickness of each wall of double plain concrete bearing walls shall be not less than 4 in. and if greater thickness is needed to carry the loads, positive means shall be provided to transmit the loads to both walls, or the thickness of the interior wall shall be increased.

(*b*) *Curtain Walls:* Exterior walls which act merely as curtain walls in a reinforced-concrete framework shall be designed to withstand a wind pressure of 25 lb. per sq. ft. on exposed surface. Reinforced-concrete curtain walls may be constructed by hand plastering and back-plastering on expanded metal or wire mesh reinforcement or by mechanical means of placing concrete or stucco.

(c) Partition Walls: Partition walls may be of plain concrete 4 in. thick cast in place or of precast units having a compressive strength of at least 500 lb. per sq. in. on the gross area of the unit or constructed by plastering and back-plastering on metal lath attached to the floor above and below.

(d) Load Carrying Precast Units: Reinforced-concrete structural units which act at the same time as load carrying elements and enclosing members shall be proportioned according to accepted engineering design to carry both vertical and horizontal loads. Exterior bearing walls of large precast units having a length equal to story height, bonded together by registering or interlocking projections and depressions, grouted in place, shall be not less than 6 in. thick, or the units shall be reinforced to carry both vertical and wind loads in accordance with accepted principles of engineering design. Large or small reinforced-concrete units connected on two opposite edges to structural members designed to carry all loads to foundations originating from the weight of the building, or from wind pressure, shall have dimensions and reinforcement determined by the bending stresses produced by wind pressure of 25 lb. per sq. ft. on exposed surface.

(e) Concrete Hollow Block, Brick or Tile: The following table shall govern the thickness of superstructure walls of concrete hollow block, concrete brick or concrete tile:

WALL THICKNESS IN INCHES FOR CONCRETE HOLLOW BLOCK, CONCRETE BRICK AND CONCRETE TILE

No. of Stories	Basement	1st Story	2nd Story	3rd Story
1	8	8	..	..
2	10	8	8	..
3	12	10	8	8

(f) Solid Concrete Exterior Walls shall be furnished with furring on the inside so as to produce an insulating air space between the interior finish and the concrete wall. Concrete walls which provide a continuous dead air space within the wall may be finished without further provision for insulation.

FLOORS

Section 4. Floors shall be designed to carry a live load of 40 lb. per sq. ft. uniformly distributed, in addition to the dead load. See Section 7.

ROOFS

Section 5. (*a*) Flat concrete roofs shall be designed to carry the dead weight of the roof and 25 lb. per sq. ft. additional for houses constructed in climates subject to heavy snowfall. Sloping roofs shall be designed for 25 lb. per sq. ft. on the vertical projections of the roof surface exposed to the wind.

(*b*) Concrete roofs without other covering shall be constructed of non-porous aggregates so graded as to produce a dense, impervious concrete. For additional assurance of watertight construction, waterproofing compounds may be used. Reinforcement to the amount of at least 0.2 per cent shall be placed in the top portion of uncovered roof slabs to resist the temperature stresses.

MATERIALS

Section 6. (*a*) Only standard portland cement, which meets the requirements of the Standard Specifications for Cement of the American Society for Testing Materials, as of Jan. 1, 1921, with subsequent amendments thereto shall be used in the construction of concrete houses.

(*b*) Aggregates shall be clean material, free from injurious amounts of dust, ashes, lumps of coal, vegetable loam and organic matter.

(*c*) Cinders may be used as a coarse aggregate for partitions and for exterior walls, providing tests show the resulting concrete will average a compressive strength of ten times the computed loads to which it will be subjected. Cinders shall be composed of hard, clean, vitreous clinker, free from unburned coal and ashes and from injurious amounts of sulphur.

(*d*) Slag used for coarse aggregate shall be clean, dense, air-cooled blast furnace slag containing not more than 2 per cent of sulphur.

(*e*) Bars used for reinforcement shall conform to the requirements of the Specifications of the American Society for Testing Materials for Concrete Reinforcing Bars in effect Jan. 1, 1921, with subsequent amendments thereto. Cold drawn steel wire, made from billets, may be used in floor and roof slabs, column hooping and for temperature and shrinkage stresses. Wire mesh or expanded metal may be used for its full cross-sectional value to resist stresses, providing its component parts meet the requirements for tests for concrete reinforcement bars of the American Society for Testing Materials.

(*f*) The water used in mixing concrete shall be free from oil, acid, alkalies and organic matter.

DESIGN

Section 7. The design of floors, roofs, beams, girders and columns shall be governed by Section 4—"Design," of the American Concrete Institute Standard Specifications No. 23.

CONSTRUCTION

Section 8. (*a*) Reinforcement shall be properly located and secured against displacement during the placing of the concrete. This is to be accomplished by some positive mechanical device locking the reinforcement in a predetermined position.

(*b*) Machine mixing is to be preferred, but where it is necessary to mix by hand, all ingredients shall be turned together until the mass is homogeneous in appearance and color. Hand mixing shall be done without losing an appreciable amount of mortar. A small batch mixer is most satisfactory. The materials shall remain in the mixer at least one minute.

(*c*) Only enough water shall be used to produce a consistency such that the concrete will flow sluggishly into the forms and around the reinforcement without separation of the aggregates from the mortar. Where the system of forms is such as to require the removal of the forms immediately after pouring the concrete, a dry mix is permissible, if the concrete is thoroughly tamped into the forms—the exterior surface of such a wall to be finished with a stucco. Concrete shall be protected against rapid drying out and shall be protected against freezing until it has hardened for at least ten days in a temperature not less than 50 deg. Fahr. Concrete shall be deposited in the forms not more than thirty minutes after mixing.

(*d*) Forms shall be substantial and sufficiently tight to prevent leakage of more than one per cent of the mortar. They shall not be removed until the concrete has hardened sufficiently to sustain without injury to the concrete the loads that will come upon it. Window and door frames may be set in the forms and the concrete cast around them. Wooden frames should be well primed and should be anchored to the concrete by means of long spikes or bolts. They should be braced against distortion from the pressure of fresh concrete.

(*e*) Back-filling on the outside of basement walls shall not be done until at least 75 per cent of the dead-load of the superstructure has been placed upon the walls. Back-filling shall be placed in layers and rammed and no large lumps or stones shall be used.

DISCUSSION.

Mr. Ferguson.

J. A. Ferguson.—The first page of this year's progress report of Committee S-5 bears two captions. The title of the Committee is "Committee S-5, on Reinforced-Concrete Houses." The caption for the progress report is "Recommended Practice for Concrete House Construction."

There is no committee of the Institute whose work is to provide complete and well organized information in connection with concrete house construction, but rather several committees all dealing in a somewhat unrelated way with different phases of the main subject. I simply wish to present here a broader plan for the work of the Institute in connection with concrete house construction. I would like to suggest that the name of this committee be changed so as to include all phases of the subject, and that it would meet a very real need of builders to draft, as quickly as may be, a "Recommended Practice for Concrete House Construction."

I wish to submit a plan to the Institute for a recommended practice for concrete house construction. This plan is intended to cover the whole subject including all standards of the Institute for any building unit entering into the construction; also a description of the various problems to be met with in house construction; a recommended method of solving these problems and any and all information that would make the report of service to architects, engineers, builders, owners, and manufacturers of building materials. The outline of this proposed report given in the following will indicate the character and amount of information which it is recommended this report should contain:

First, the report should be entitled "Recommended Practice For Concrete House Construction."

Section 1 should contain a statement of the scope of the report together with definitions of the several types of dwelling houses, the standard practice for the construction of which is recommended in the report.

Section 2 should contain a carefully worked out essay upon the planning of the home; this section should be drafted by architectural experts, illustrated by plans and sketches and criticisms or suggestions offered by all who are familiar with the subject. It should be complete, illustrating all forms of construction and efficient methods as well for the usefulness and comfort derived from the several types of buildings as for their practicability and reasonableness of cost.

This essay should also contain a complete description of the methods by which beauty and grace are to be obtained, monotony and sameness of repeated structures prevented and how a distinctive type of concrete dwelling houses may be developed that will be attractive and pretty, while at the same time have a reasonable cost. At this point the speaker would like to state that so far the ingenuity of architects has developed very few houses, which to his eye would make a pleasing home. Invariably the form of building is plain, rectangular, barren of line and ornament

Mr. Ferguson. except for certain rude and really ugly features. I know that pretty and attractive concrete houses can be designed by the architect because they have done it. In view of this I think it is highly important that such a report as I am trying to outline here should contain a complete well worked out architectural statement with illustrations to convey the matter contained in the written description.

Section 3, Structural Design, should contain a complete description and specification as far as it is necessary to carry it out for purposes of dwelling house construction.

Section 4, Materials of Construction, should contain all the standards of the Institute with special reference to the usefulness of all materials, systems, units and forms of construction to be used in dwelling houses.

This will include a section on both bearing and non-bearing wall units, both reinforced and non-reinforced; standard requirements for which should be so well planned and arranged that they can be adopted throughout the country as standards for architects' specifications and as information for manufacturers of materials.

Section 5, Chimneys, Flues and Heating Apparatus, should contain full and complete information as to the proper method of installing chimneys, flues and heating apparatus; descriptive details and illustrations will be useful here as elsewhere.

Section 6, Building Accessories and Appurtenances, should contain full information as to how to place hot water piping, warm air flues and registers, electric wiring, plumbing, suspended ceilings, concrete gutters, and how to place furring on the wall and many other similar subjects.

Without enumerating further subject matter for such a report, you will realize what I am proposing means a great deal of work and that while the report would be lengthy and full of detail, yet I am positive that some such information is necessary before the wonderful possibilities inherent in concrete construction for dwelling houses will be either understood or realized; such information will certainly be found useful and would promote the use of concrete materials in the construction of comfortable, pleasing and attractive homes.

REPORT OF COMMITTEE S-6, ON CONCRETE ROADS.

(Informally presented by C. R. Ege, Secretary.)

The committee is not in position to make a formal report at this time. It is rather a large one and widely distributed so that it has been practically impossible to call the members together for a meeting, and the progress by means of correspondence has been unfortunately slow. The committee has to bring our recommended practice for the construction and design of concrete pavements, and also the specifications, up to date. As far as the specifications are concerned, there are not very many details which need revision. The present specifications were adopted by letter ballot of this Institute in 1918, and there are only a few matters which require revision. In the recommended practice for pavement construction there are quite a number of details which should be brought up to date. There is a tremendous amount of research work being done, and it should be our job to bring the results of that work together and make it available for the entire industry.

The interest being taken in research connected with concrete pavements now is nationwide, and there are some very important undertakings underway, notably the Bates road in Illinois, the U. S. Bureau of Public Roads experimental road near Arlington, Va., and the Pittsburgh testing track. With all of these tests nearing completion this spring, we will have a large amount of real information on which to base future recommendations.

In the past two years there have been a good many important developments in connection with the utilization of aggregates. It was formerly supposed that we had to use aggregates of a certain size and grade, and if those aggregates were not available we could not build concrete roads. The work that Prof. Abrams has done, and, based on his investigations, the work that has been done by the various state highway departments, has shown how we may build perfectly satisfactory concrete pavements using such aggregates as are economically available. In other words, we should be able to take the experience of those states now and tell an engineer or contractor how he may build a good road with the aggregates that he has economically available in his vicinity.

It is possible we may be able to offer specifications and revised specifications and recommended practice for letter ballot before the next annual session of the Institute.

DISCUSSION.

Ernest Ashton.—I would like to ask Mr. Ege, in his mention of the suitability of aggregates, whether the investigation also includes the relative hardness of the aggregates? It has been found in some experiments, that a high coefficient does not necessarily imply that the aggregate is good. It is quite possible, therefore, that some of the banks we have turned down might be utilized by using softer aggregates in the proper proportions. **Mr. Ashton**

C. R. Ege.—That is being brought out by the experience in some of the states. There are several states in the Mississippi Valley in which the only aggregates are limestone, which may go as low as five or six on the scale of hardness, and there have been developed methods of proportioning and constructing which will yield a very satisfactory pavement. The experiments of the Lewis Institute have shown that the proportion and control of consistencies have much more to do with the resistance of the pavement to abrasion than the construction and hardness of the aggregate itself, particularly the coarse aggregate. **Mr. Ege.**

H. S. Van Scoyoc.—I would like Mr. Ege to give us a few words about double-slab construction. **Mr. Van Scoyoc.**

Mr. Ege.—In a number of states, particularly those along the North Atlantic Coast, there has been developed in the past eighteen months a type of construction with a longitudinal joint or plane of weakness. One point of using that form of construction was to control the danger of longitudinal cracking or at least to control the location of the break. I think perhaps that form of construction was undertaken or specified before anybody fully realized just why it might be the thing to do. In developments which have been observed on the experimental roads it has been found that there is a movement of the slab every day due to temperature. In other words, the edges of the slab will curl downward during the day due to the difference in temperature between the top and bottom of the slab. During the day the surface of the slab is warmer than the bottom of the slab and expands slightly more. That has a tendency to turn those outer edges of the slab downward. At night the reverse is true and the edges of the slab come up, and may come up so far as to separate themselves entirely from the sub-grade. The total amount of movement of the edges of the slab may be as great as one-tenth of an inch. **Mr. Ege.**

This makes it very clear why in some cases we have had longitudinal cracks in concrete pavements which could not altogether be explained. Mr. Older has found that where he has a slab with an existing longitudinal crack or joint approximately in the center of the slab, the movement of the edges of the slab due to temperature is greatly reduced; it may be not more than one- or two-hundredths of an inch instead of a tenth. Consequently it appears that there is a logical reason for placing a plane of weakness or a joint in the center of a slab which is 16 or 18 ft. wide.

Mr. Ege. Another reason for the center joint is to permit the building of the road half at a time, which is very important in places where traffic cannot be detoured at the point of construction. In New York, Connecticut or Pennsylvania a detour would mean throwing traffic around a great many miles of country. This double-slab construction has to be under the control of traffic police or watchers placed there by contractor, but it is a minor inconvenience compared with having to detour traffic around a great many miles of country. It has been tried in a great many places and it adds only a small amount of cost to the cost of building the pavement.

REPORT OF COMMITTEE P-4, ON CONCRETE STAVES.

The committee on concrete stave standards has to report progress only to extent of the following statement of the objectives toward which the work of the committee is directed. These are:

(1) Determination of standard of quality of staves.

(2) Formulation of specifications covering methods of manufacture.

The chairman would state that he has found difficulty in arriving at any suitable basis for standardization. The variety of purposes to which stave construction is being applied involve a very wide range of requirements. This might be evidenced by illustration of an extreme case of tank construction where the strain imposed upon the staves by the hooping amounts to over 1300 lb. per inch of height of stave, while in all ordinary farm silo work, hoop strains are rarely more than ten or twelve percent as much. The quality of stave successfully to meet such a condition, as the tank above mentioned, will, of course, also depend upon the thickness of the stave used.

The minimum requirements as to accuracy of size and shape will differ also with respect to the type of interlock employed. Again, with respect to absorption, resistance to abrasion and other qualities, wide variations in requirements would be imposed by the various purposes for which staves are needed.

It has been taken for granted, however, that everyone engaged in the manufacture of stave structures will make, for reasons of economy and simplicity in manufacture, a single grade of stave as his individual standard. It will be further agreed that such staves should meet certain minimum requirements. It follows that staves of varying merit may come into competition in some cases on work requiring special strength. The chairman has proposed that the thought of the committee be devoted to question of specifying a limitation of size of any silo vessel in the various sorts of services for which staves may be used, which it would be considered permissible to construct with the stave of standard qualities. This policy, in other words, may be stated as follows: Where specially severe conditions of superimposed weight, or high hoop strains demand it, a special stave, better than the standard, would be offered.

The firm with which the chairman is associated has built several elevated water tanks, in some of which the actual loads carried to the ground by staves run up to 200 lb. to the square inch. It is felt that owing to imperfect end bearing of staves and, also, the fact that stave walls are thin and impossible to construct precisely vertical throughout, that the stave manufacturers generally should recognize the importance of building, in such special cases, with a unit that represents the very highest obtainable qualities. If this policy is not followed, there may be failures of stave structures that will work to the detriment of all engaged in the business.

There have been tests of staves made at the Lewis Institute under direction of Professor Abrams and at the factories of the chairman's firm to a total number of one hundred and three staves. To-day it appears impossible to draw any but the most general conclusions from the results of these tests. We have proved to our own satisfaction the merit of careful restriction of use of water in the mixture where the wet cast method is employed; we have also proved the merit of sprinkling the staves for a period of a week or more, where either the dry tamp or wet cast system is employed.

The method of testing which gives information of the largest general value is considered to be the transverse break test. In making comparisons of staves tested by this method, the basis for comparison can be only the modulus of rupture; that is, the extreme fiber stresses produced in the stave. For a given condition of loading and length of stave, the resulting modulus of rupture is, of course, dependent upon width and thickness of the stave.

In attempting to draw conclusions from results of tests above referred to, extremely wide variations are noticed ranging from less than 200 lb. per sq. in. to over 1000 lb. per sq. in. of extreme stress on concrete. It is interesting to note that these extreme figures are both recorded with staves made by wet cast systems.

Tentatively, it is considered that a stave, to be satisfactory for general purposes, should carry a minimum of 750 lb. load concentrated at the center of a 28 in. span; the staves being 10 in. wide and 2½ in. thick. This condition calls for modulus of rupture of 500 lb. per sq. in.

The chairman hopes to be able to report to the next convention progress for agreement on the policies suggested above, with further information regarding conclusions that may be drawn from systematic testing of staves at his factories or elsewhere.

WALTER BRASSERT, *Chairman.*

REPORT OF COMMITTEE P-5, ON FIRE RESISTANCE OF CONCRETE BUILDING UNITS

Is the concrete block a satisfactory unit for the construction of first class buildings? A good deal of argument has been going on between architects, building commissioners, boards of fire underwriters, builders, manufacturers of concrete block and others, but up to the present time no conclusive tests have been made or data presented to guide those interested in coming to a definite conclusion. It is true that there are no similar tests or data on other material, such as brick, lumber, stone, etc., but these materials have been in use for a far longer time and the long experience thus gained of their behavior under load and under fire conditions have guided architects, builders and others in their use and has enabled fire insurance companies to fix their insurance rates.

The concrete block is a comparatively new material and there is still a good deal of uncertainty in the minds of those interested in building construction as to its strength and fire resisting ability. In the absence of tests, the failure of poor block in fires has cast doubt upon all block and building tile units in the minds of architects, insurance underwriters and building commissioners and in many cities a burden of expense is laid upon building construction, because architects will not specify, and building commissioners will not authorize, the use of concrete block; even though their refusal to do so closes the market to a local industry, using local materials, and high-priced substitutes have to be transported from a distance, thus adding also to the burden of railroad freight congestion and increasing the amount of freight charges (which act as a tax upon towns which are situated far from the source of supply of substitute materials).

Fire insurance underwriters in many cases will not grant to concrete block buildings any better rates than those granted to frame units and although this difference in insurance rates is an insignificant annual charge on a householder, its psychological effect is out of all proportion to the amount of money involved. The fact that insurance boards will not recognize it is sufficient to bar it in many cases.

Manufacturers of concrete block have therefore experienced, in many cities, difficulty in introducing and selling their products for use in residence, warehouse and other construction. In fact the chief market for the concrete block today is in the construction of cellars, garages and other unimportant structures.

Building commissioners are often hostile to concrete block because of their alleged low strength and high absorption, but in a few leading cities, such as Pittsburgh, building authorities recognize block, made according to the specifications of the American Concrete Institute, capable of withstanding a test load of 1000 lb. per sq. in. of gross area and an absorption test not exceeding 10 per cent and have permitted their use in any structure subject to passing this test.

The fire insurance authorities, however, are still doubtful of the fire resistive merits of the concrete block and are in some cities actively opposing the revision of building codes favorable to concrete block and will also not grant to insurers of concrete block structures rates better than those given to frame construction.

In view of the fact that the methods of manufacturing concrete block vary widely, the quality and characteristics of the aggregate differ, methods of mixing, tamping, and curing are not standard, underwriters have no data to guide them in giving an equitable rating to concrete block construction. Until the fire-resistive qualities of concrete block have been determined by test the fire insurance companies do not feel justified in giving to this comparatively new material the rates which experience has guided them in granting to brick, stone and other forms of masonry construction.

In order to remove this objection and establish definite standards to guide insurance companies in their insurance ratings and to gain from them recognition of the fire-resisting qualities of concrete block, a comprehensive series of tests will be necessary.

The matter has been discussed with the Underwriters Laboratories (by whose tests and conclusions the local boards of fire underwriters are largely influenced), and a program of tests has been arranged from which it will be possible to ascertain from a fire protection viewpoint the value of the construction made of the materials submitted and to determine the nature of the recommendations to be made.

The investigation is in the nature of a study of samples representative of the products of the industry as a whole and does not contemplate definite findings concerning the products of any individual manufacturer. If it is carried through it should form a basis for a report in which the classification of hollow concrete block from the fire-retarding point of view will be established.

Physical properties of representative samples from lots selected for the tests will be determined in accordance with the adopted standard program of the American Concrete Institute for the test values in question, these being strength in compression, of specimens dried to constant weight, of specimens completely saturated, of specimens saturated and re-dried, of specimens which have been subjected to freezing and thawing, and later of samples removed from fire test panels subjected to the standard fire endurance test and to the fire and standard hose stream test, absorption test and freezing and thawing test, also tests covering adhesion of mortar.

Members of the committee will supervise the process of manufacture of the sample block shipped for use in test, and will identify the individual samples so shipped as to method of manufacture, character of mix and other items having a bearing upon test performance as well as to date of manufacture.

It is proposed that in all of the tests, unless otherwise noted, the block tested is to be of the three-core oval type 8 in. by 8 in. by 16 in., to be of 1: 5 mix, to be of a semi-dry mix having a consistency wet as possible, tamped with automatic tampers and to be hardened in the steam room. Except as otherwise specified the aggregate is to be Elgin sand and pebbles grading 0 to ¾ in.

It is not intended in this first series of tests to include concrete brick or special block of any type or units made by the Zagelmeyer or Hydrostone process. After viewing the results of this first series of tests, tests on these may be made the subject of a second series.

Panel A.—Using Furnace No. 2. Panels to be mounted in a steel frame providing exposed face of panel approximately 10 by 11 ft., providing therefore 10 courses of block, each course containing approximately 7½ block. This panel to be tested up to time of failure, but not to exceed 5 hours, the furnace temperature to be in accordance with the standard time temperature curve. At the end of 5 hours the panel to be withdrawn from the furnace and thereupon subjected to the standard fire hose stream test.

Panel B, C. and D to be similar in all respects to Panel A, except that the section of the block for Panel B is two-core square, for Panel C is one-core rectangular, and for Panel D a concrete structural tile 5 in. x 8 in. x 12 in. is used. Panels B, C and D will be exposed in Furnace No. 2 to the standard fire conditions in accordance with the standard time temperature curve for a minimum period of one hour and for a maximum period of 5 hours. The actual time of exposure being determined at the time of test according to observations made of the condition of the face of block exposed to fire and in accordance with the judgment of the test engineers as to the best period for discontinuing test in order to ascertain the exact relative performance.

It is not proposed to subject Panels B, C and D to the fire hose stream test.

In addition to the four tests listed in the foregoing, four additional tests in full size panels in Furnace No. 2 are to be run. These tests are to bring out differences, if any, in performance in fire due to the several variations in manufacture or composition of block as indicated in the following.

Panel E will be assembled as follows: Courses 1 to 5 from the bottom of the panel to be filled with block of a semi-dry consistency. Courses 6-10 to be filled with block made with a damp consistency. Courses 11 to 16 to be filled with block mixed as wet as possible. All of these block to have been hardened in the steam room, to be of a 1: 5 mix, to have an aggregate of Elgin sand, with 0 to ¾ in. pebbles, and to be of the three-core oval type.

Panel F to be filled with block in the manner indicated for Panel E, the various details being the same, except that all block has been hardened in air instead of the steam room.

Panel G is to be assembled similar to Panels E and F, except that the three sets of courses will be filled with block representing variations in mix as follows:

Courses 1 to 5—a mix of 1 : 3
Courses 6 to 10—a mix of 1 : 4
Courses 11 to 16—a mix of 1 : 6

In other respects all of the block in this Panel G to be identical. In other words, to be three-core oval blocks 8 by 8 by 16 in.; aggregate consisting of Elgin sand and pebbles graded 0 to ¾ in.; consistency wet as possible and hardened in steam room. Note that the corresponding block having a mix of 1 : 3 has been previously tested.

Panel H to be run to determine the effect of fire on different kinds of aggregates, the panel to be filled with four groups of block each in four courses. For courses 1 to 4 block shall have an aggregate of limestone screenings and crushed limestone. Courses 5 to 8 block shall have an aggregate of Elgin sand and cinders, grading 0 to ¾ in. For courses 9 to 12 the block shall have an aggregate of Elgin sand and crushed slag. For courses 13 to 16 the block shall have an aggregate of Elgin sand and silicious gravel. In all other tests an aggregate consisting of Elgin sand and pebbles grading 0 to ¾ in. has been employed and in at least one of the previous tests a combination employing this aggregate and in other respects identical with the block to be tested in this series "H" is used. Panel H will be placed in the furnace first.

In panels E, F, G and H, steel plates ⅜ in. thick and about 6 in. wide will be inserted in the mortar joints, between the courses where block differ. This steel plate is a precaution to avoid interference with performance in test of block in one set of courses in the event of unfavorable performance of block in adjacent courses.

During all of the foregoing tests careful observations will be made in connection with the installation of the samples in the test panels, in connection with the conduct of the test of the wrecking of the panels after test, of the dimensions, appearance, workmanship shown, characteristic qualities of each individual block in all of the various groups, representative samples to be selected from each panel or from each group of block in a single panel and subjected to crushing and absorption tests afterwards to determine the effect of fire upon those physical properties.

Estimates of the cost of these tests have been made as follows:

Fire tests at Underwriters Laboratories	$5000 to $6000
Crushing and absorption tests at Lewis Institute	500
Inspection of block manufacture at various yards, freight and other miscellaneous expenses	500
Total	$6000 to $7000

These tests will be of inestimable value to the large and rapidly growing concrete products industry and remove one of the hindrances to its rapid expansion.

It is the desire of your Committee that this series of tests be approved by the Institute and put in hand as promptly as possible and with this purpose in view wishes to raise funds for the project. Ways and means have been discussed. The Concrete Products Association is raising a fund to bear a share of the expense. The Portland Cement Association is willing to contribute and other efforts are being made to raise a fund sufficient to cover these tests and it is believed that with Institute endorsement and co-operation the funds will be quickly and easily secured.

HARVEY WHIPPLE, *Secretary*
LESLIE H. ALLEN, *Chairman*

[After the presentation of the report the following resolution was adopted by the meeting.]

FIRE RESISTANCE OF CONCRETE BUILDING UNITS
RESOLUTION PASSED FEB. 14, 1922.

Be It Resolved:

1—That the report of Committee P-5 on Fire Resistance of Concrete Building Units, be approved that the committee be continued.

2—That the committee be authorized to raise the necessary funds and as agents for the American Concrete Institute be authorized to contract with the Underwriters Laboratories, Inc., of the National Board of Fire Underwriters for the carrying out of the fire tests at a cost not exceeding $6000; and also carry out the scheduled compression and absorption tests at Lewis Institute, provided that funds for these tests be subscribed in full before such contract is made.

3—That the members in convention here assembled endorse the request of this committee that the Board of Directors be asked to appropriate a sum of not less than $1000 as the Institute's portion of the general fund, and that this money be used for making the tests together with money obtained from other sources.

4—That the committee be authorized to invite the Portland Cement Association; the Concrete Products Association, and the Concrete Block Machinery Association to subscribe to and to co-operate in these tests, and that similar invitations be also extended to any other association, or society, that may be suggested by the Executive Committee of the Institute, or by members of Committee P-5.

REPORT OF COMMITTEE P-6, ON CONCRETE PRODUCTS PLANT OPERATION.

From year to year as the activities of the American Concrete Institute expand, they come more and more in contact with a concrete industry that has been struggling along practically unaided by scientific research. Every step of its progress has been contested by strongly combined competitors. Regardless of all of this opposition, that industry, the concrete products industry, has interested thousands of investors who have put over fifty millions of dollars into factories making various kinds of products.

Ordinarily, concrete products are considered as concrete block, drain tile and pipe, but these are only a few of the hundreds of products now being manufactured. The possibilities are unlimited as is readily shown by the fact that several concrete products manufacturers are now specializing on furnishing traceries for church windows.

Your committee, without knowing what are its duties, is greatly impressed with the responsibility you have put upon it. It realizes that there are limitations to the extent to which the American Concrete Institute can be interested in concrete manufacture, and it is afraid that it might overlook some assistance that can be rendered. If, therefore, some of its recommendations are beyond the scope of the Institute's activities, it may at least suggest some phases on which the Institute can act now or in the future.

Though it is true that all kinds of concrete products are being manufactured, a close investigation of the concrete products industry would show that the advancement in concrete products manufacture has been very slow. What is needed most is scientific research and the application of engineering principles combined with more enlightened management and sales methods.

The first problem in the design of the products plant is the general layout. Though local conditions, such as availability of aggregate, trackage and output, will affect the design materially, certain features of plant layout should ordinarily be incorporated depending upon the output. Mechanical handling of the aggregates and cement into elevated bins so that gravity can be used in plant operation is desirable.

A study of plant layouts has resulted in the preparation of many plans, drawings of which are herewith attached. These show not only the detailed layout, but also a daily operation chart.

In studying the design of curing rooms for concrete products plants, it is evident there is not so much trouble in arriving at a conclusion as to the best design because the problem is to give moisture and heat to the product at the least cost in order that the hardening of the product can be hastened.

Though several methods have been suggested at various times for

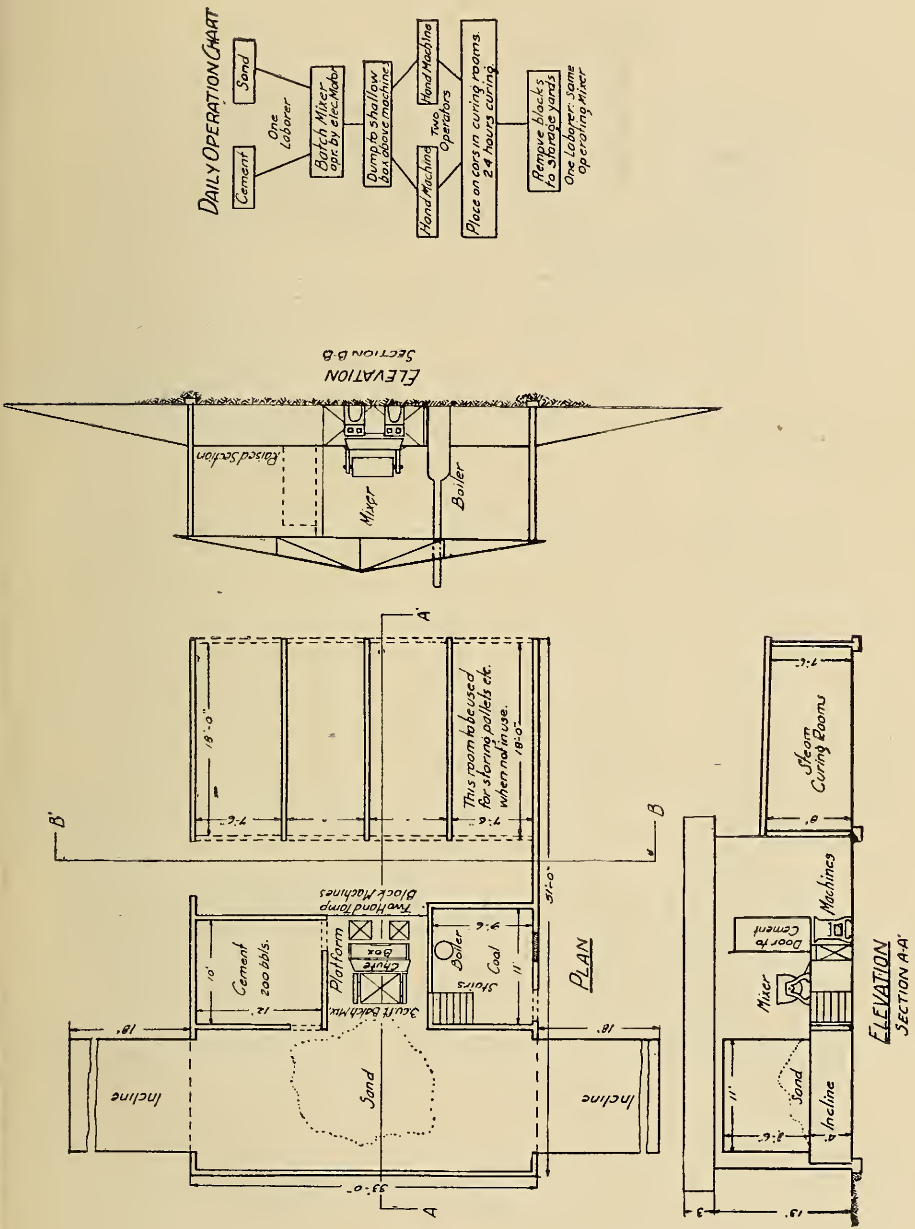
DAILY OPERATION CHART
Cement
Sand
One Laborer
Batch Mixer opr. by elec. Motor
Dump to shallow box above machines
Hand Machine
Hand Machine
Two Operators
Place on cars in curing rooms. 24 hours curing.
Remove blocks to storage yards
One Laborer: Same operating Mixer
ELEVATION
SECTION B-B
Raised Section
Mixer
Boiler
A'
B'
B
18'-0"
7'-6"
This room to be used for storing pallets etc. when not in use.
18'-0"
7'-6"
Two Hand Tamp Block Machines
Cement 200 bbls.
10'
12'
Platform
Chute Box
3 cu. ft. Batch Mix.
9'-6
Boiler
Coal
11'
Stairs
51'-0"
18'
18'
Incline
Incline
Sand
33'-0"
A
PLAN
7'-6"
Steam Curing Rooms
8'
Machines
Door to Cement
Mixer
Sand
Incline
11'
8'-6"
4'
3'
19'
ELEVATION
SECTION A-A'

FIG. 1.

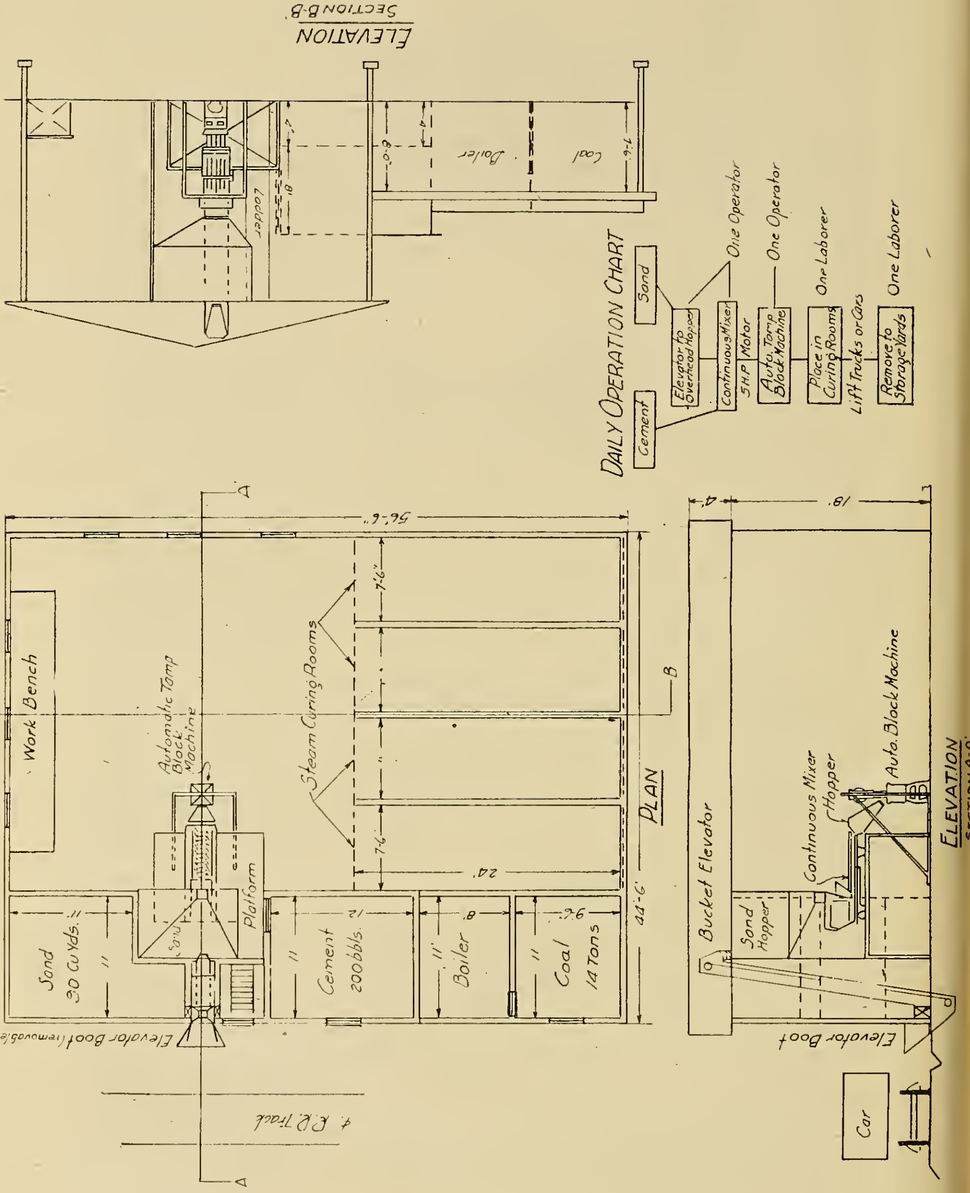
ELEVATION
SECTION B-B'
7'-6"
8'-0"
4'
7'
18'
Ladder
Boiler
Coal
DAILY OPERATION CHART
Sand
Cement
Elevator to Overhead Hopper
Continuous Mixer
5 H.P. Motor
Auto. Tamp Block Machine
Place in Curing Rooms
Lift Trucks or Cars
Remove to Storage Yards
One Operator
One Operator
One Laborer
One Laborer
A
A'
B
B'
56'-6"
44'-6"
Work Bench
Automatic Tamp Block Machine
Steam Curing Rooms
7'-6"
7'-6"
24'
Platform
Sand
Sand 30 Cu. Yds.
11'
11'
Cement 200 bbls.
12'
Boiler
8'
Coal 14 Tons
9'-6"
Elevator Boot (removable)
℄ R.R. Track
PLAN
4'
18'
Bucket Elevator
Sand Hopper
Continuous Mixer
Hopper
Auto. Block Machine
Elevator Boot
Car
ELEVATION
SECTION A-A'

FIG. 2.

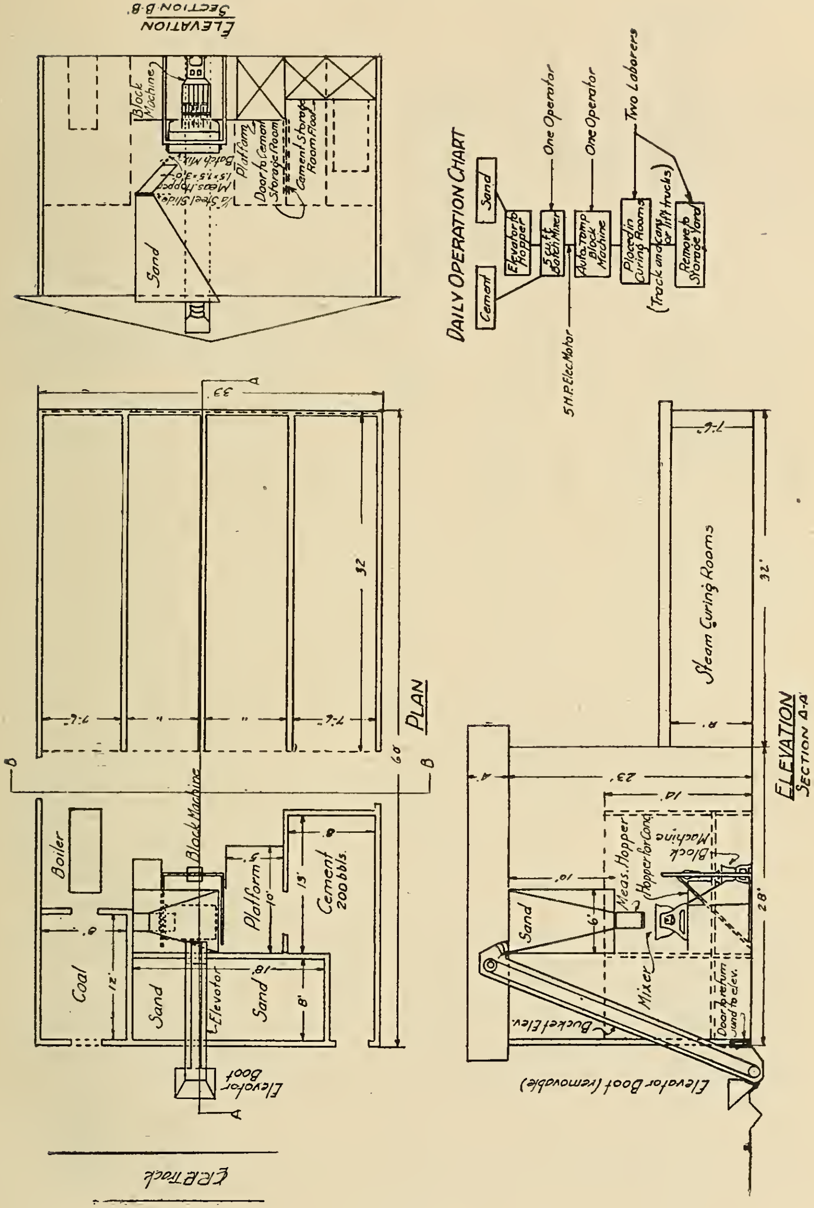
ELEVATION
SECTION B-B'
Block Machine
Platform
Door to Cement Storage Room
Cement Storage Room Floor
Batch Mix
Meas. Hopper 1.5'×1.5'×3.0'
1/2 Steel Slide
Sand
DAILY OPERATION CHART
Cement
Sand
Elevator to Hopper
5 cu. ft. Batch Mixer
Auto. Tamp Block Machine
Placed in Curing Rooms
Remove to Storage Yard
One Operator
One Operator
Two Laborers
5 H.P. Elec. Motor
(Track and Cars or lift trucks)
PLAN
33'
32
7'-6"
60'
B
B
A
A
Boiler
Coal
Sand
Elevator
Sand
Platform
Cement 200 bbls.
Block Machine
Elevator Boot
12'
6'
18'
8'
10'
13'
5'
8'
C.R.R. Track
ELEVATION
SECTION A-A'
Steam Curing Rooms
7'-6"
32'
8'
23'
14'
4'
10'
6'
28'
Sand
Meas. Hopper
(Hopper for Conc.)
Block Machine
Mixer
Bucket Elev.
Door to return sand to elev.
Elevator Boot (removable)

FIG. 3.

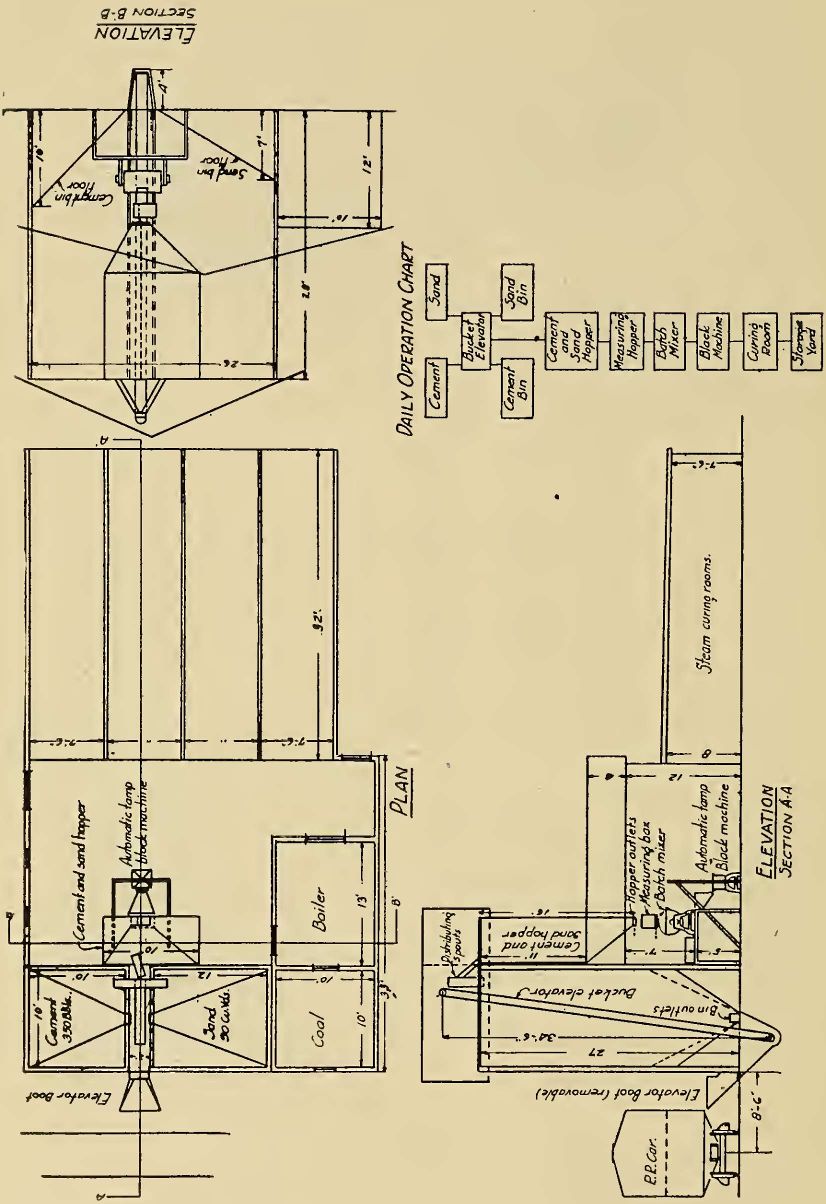
ELEVATION
SECTION B-B
Cement bin floor
Sand bin floor
4'
10'
7'
12'
10'
28'
26'
DAILY OPERATION CHART
Cement
Sand
Bucket Elevator
Cement Bin
Sand Bin
Cement and Sand Hopper
Measuring Hopper
Batch Mixer
Block Machine
Curing Room
Storage Yard
A
92'
7'-6"
7'-6"
Steam curing rooms.
PLAN
Cement and sand hopper
Automatic tamp block machine
Boiler
13'
8'
Coal
10'
10'
3.3'
Cement 350 Bbls.
Sand 90 Cu. Yds.
10'
10'
12'
Elevator Boot
A
Distributing Spouts
Cement and Sand hopper
11
16'
Bucket elevator
Bin outlets
34'-6"
27
7
3
12
8
Hopper outlets
Measuring box
Batch mixer
Automatic tamp
Block machine
ELEVATION
SECTION A-A
Elevator Boot (removable)
R.R. Car.
8'-6"

FIG. 4.

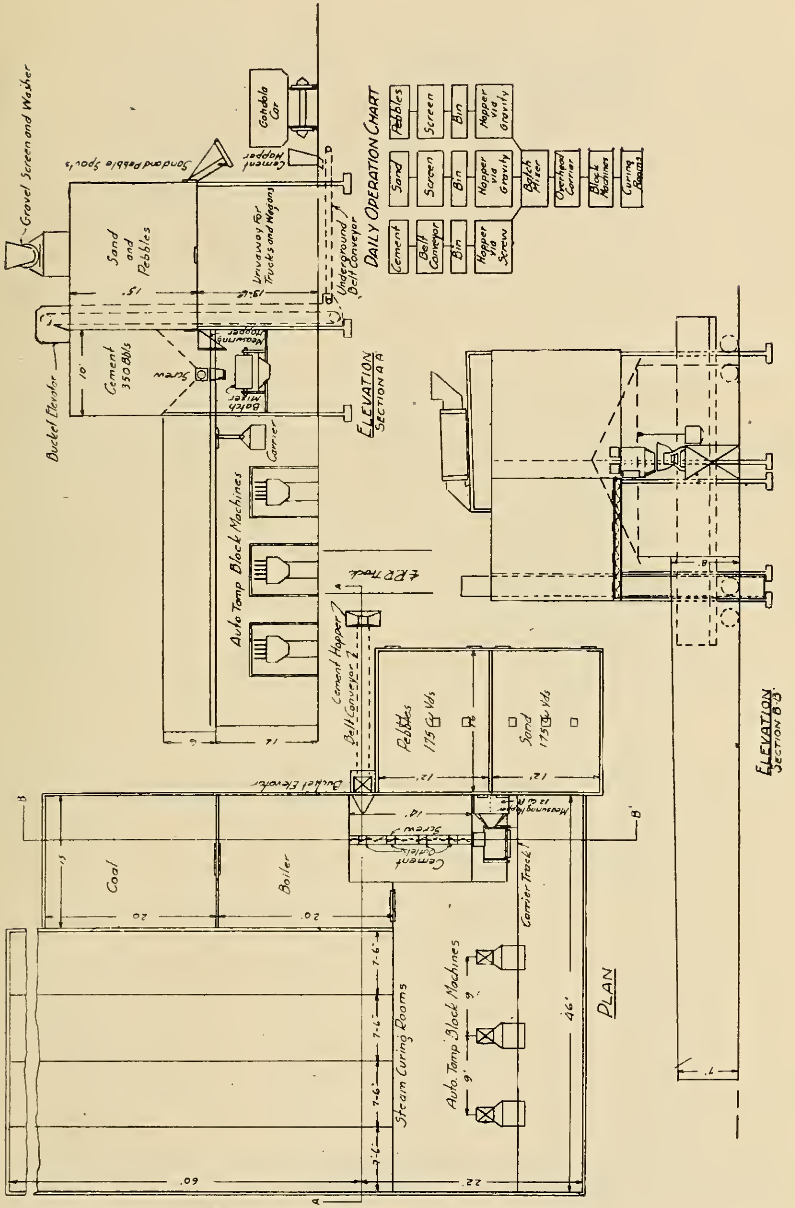
Gravel Screen and Washer
Sand and Pebble Spouts
Gondola Car
Cement Hopper
Sand and Pebbles
Driveway for Trucks and Wagons
Underground Belt Conveyor
Bucket Elevator
Cement 350 Bbls
Screw
Measuring Hopper
Batch Mixer
Carrier
Auto Tamp Block Machines
ELEVATION SECTION A-A
DAILY OPERATION CHART
Cement
Belt Conveyor
Bin
Hopper via Screw
Sand
Screen
Bin
Hopper via Gravity
Pebbles
Screen
Bin
Hopper via Gravity
Batch Mixer
Overhead Carrier
Block Machines
Curing Rooms
R.R. Track
Cement Hopper
Belt Conveyor
Pebbles 175 Cu Yds
Sand 175 Cu Yds
Bucket Elevator
Measuring Hopper
Screw
Cement Outlet
Coal
Boiler
Carrier Track
Steam Curing Rooms
Auto. Tamp Block Machines
PLAN
ELEVATION SECTION B-B

FIG. 5.

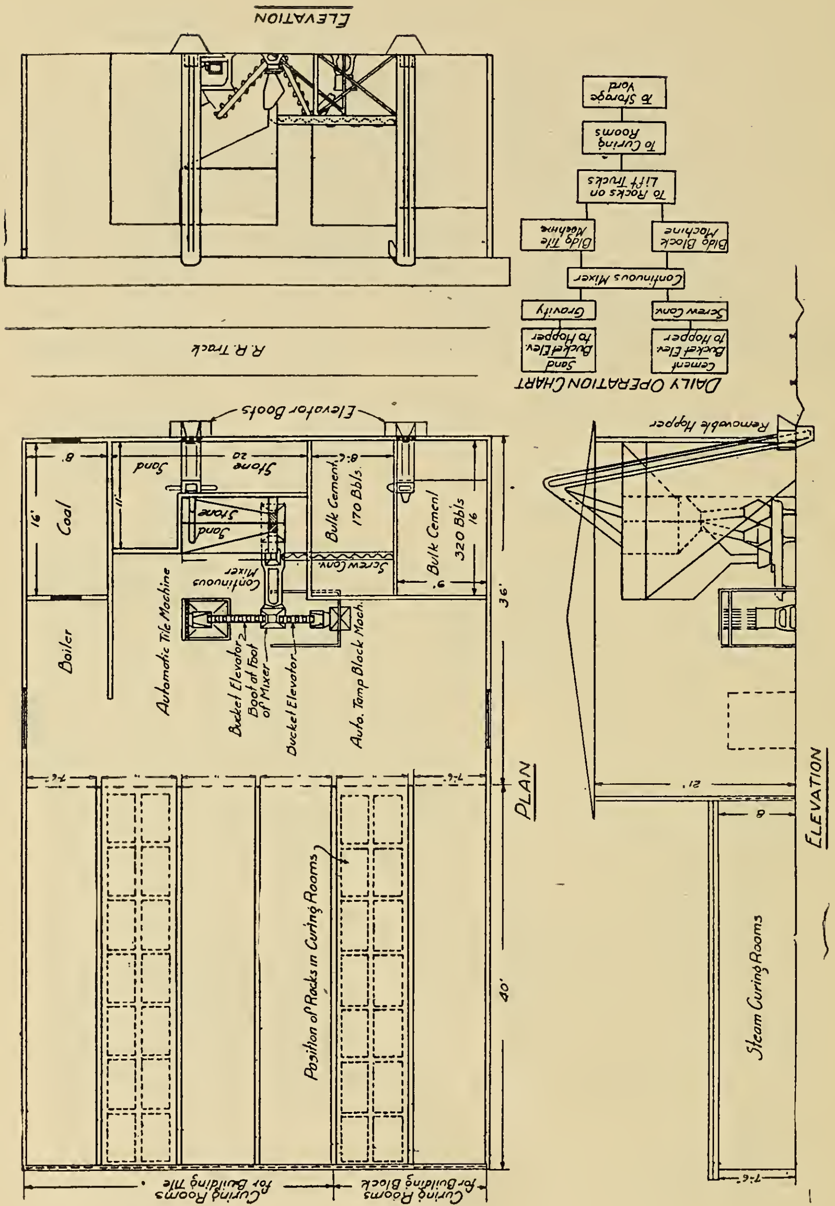
ELEVATION
R.R. Track
Elevator Boots
8'
Sand
20
Stone
8'-2"
16'
Coal
11'
Stone
Sand
Bulk Cement 170 Bbls.
Bulk Cement 320 Bbls
16
Screw Conv.
Continuous Mixer
6'
36'
Boiler
Automatic Tile Machine
Bucket Elevator
Boot at Foot of Mixer
Bucket Elevator
Auto. Tamp Block Mach.
7'-6"
7'-6"
PLAN
Position of Racks in Curing Rooms
40'
Curing Rooms for Building Tile
Curing Rooms for Building Block
DAILY OPERATION CHART
Sand Bucket Elev. to Hopper
Cement Bucket Elev. to Hopper
Gravity
Screw Conv.
Continuous Mixer
Bldg Tile Machine
Bldg Block Machine
To Racks on Lift Trucks
To Curing Rooms
To Storage Yard
Removable Hopper
21'
8
Steam Curing Rooms
7'-6"
ELEVATION

FIG. 6.

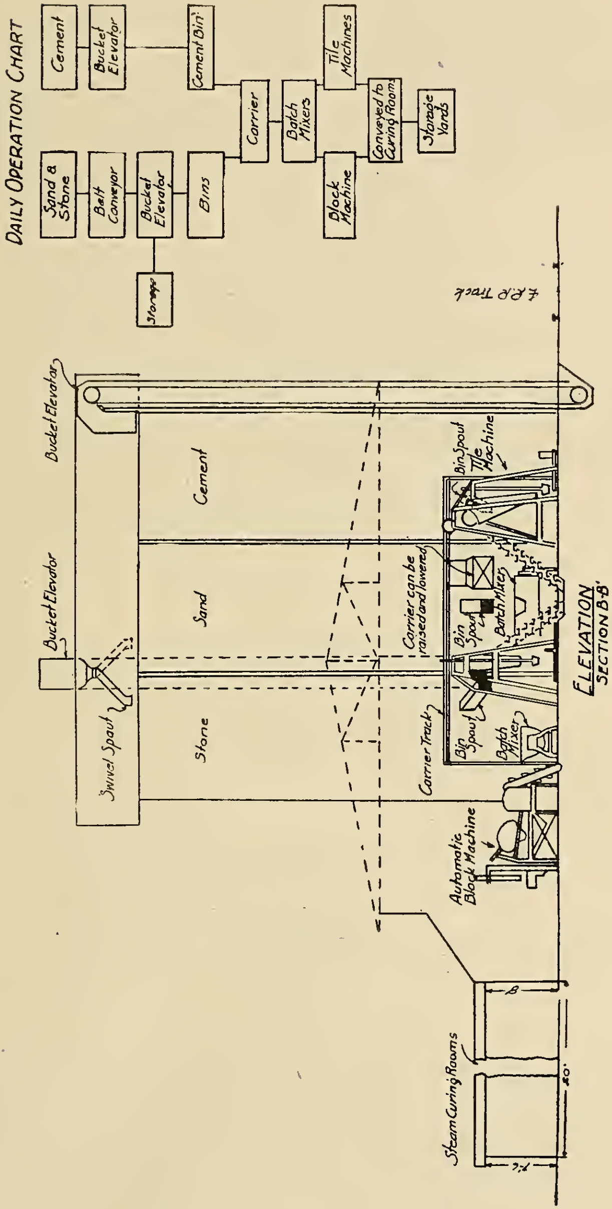
DAILY OPERATION CHART
Sand & Stone
Belt Conveyor
Bucket Elevator
Storage
Bins
Cement
Bucket Elevator
Cement Bin
Carrier
Batch Mixers
Tile Machines
Block Machine
Conveyed to Curing Rooms
Storage Yards
℄ R.R. Track
Bucket Elevator
Bucket Elevator
Swivel Spout
Cement
Sand
Stone
Carrier can be raised and lowered
Carrier Track
Bin Spout
Tile Machine
Batch Mixers
Bin Spout
Bin Spout
Batch Mixer
Automatic Block Machine
Steam Curing Rooms
8'
20'
12'
ELEVATION
SECTION B-B'

FIG. 7.

steam curing rooms, such as the use of steam shot into troughs of water, radiator systems and water sprinkling and low pressure steam shot directly into the curing rooms, a scientific analysis from the standpoint of effectiveness and economy shows that steam radiation through pipes with an extra pipe equipped with controlled openings to regulate the amount of saturation in the curing room will prove most satisfactory.

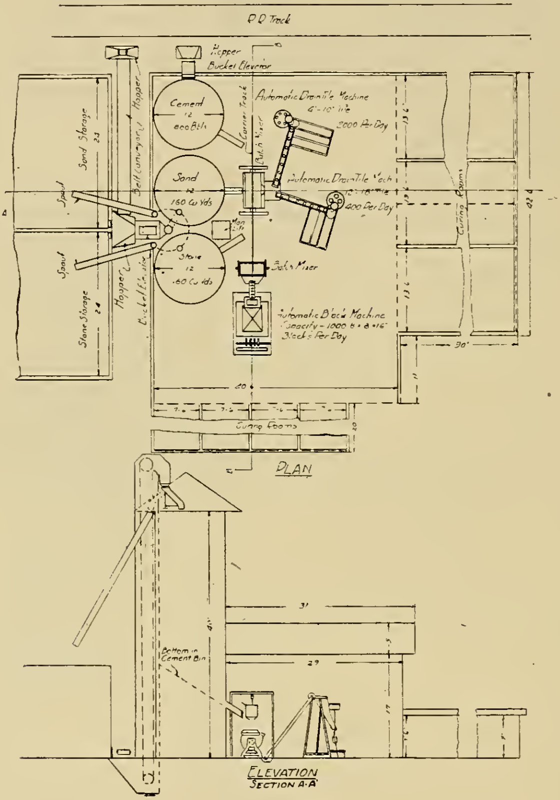

FIG. 8.

Before designing the heating system, climatic conditions should be studied to arrive at the temperatures to be used. Obviously, a heating plant designed to meet the extreme conditions of time and temperature would be entirely too large for normal conditions. In a temperate climate, there may be several days of winter with a temperature down to zero, or even below. The temperature of most of the winter days would be at least 32° F. It must also be kept in mind that winter conditions prevail not over three months, or 25 per cent of the time. Mild and warm weather covers the remainder of the 75 per cent. In warm weather the factory output should be at a maximum, and therefore the curing time should be at the minimum. It is during this time that the finished product may be required to be ready for outside storage in 24 hours.

From the best information available, the maximum rate of absorption of heat by wet concrete in still air is approximately 1½ B. T. U. per square foot of exposed surface per degree difference in temperature per hour. Therefore, knowing the number of products in a chamber, their exposed area and the number of pounds of concrete the total B. T. U. required to raise the product to a given temperature and the number of hours required to absorb that heat can be determined.

Another factor in the problem is the quantity of heat generated in the hardening of concrete. Although some experiments have been conducted indicating that considerable heat is given off through chemical action, results are not such as to make it possible to state definitely how many B. T. U. should be figured under ordinary operating conditions.

It is evident, however, that this quantity of heat will help materially. As there are, however, many unknown factors in the problem, this extra amount will only be considered as being useful when the temperatures are outside the range for which the heating system is designed.

DESIGN ASSUMPTIONS.

The following assumptions are made in designing the plant:

1. Production, 1000 8x8x16 in. block per day.
2. Each curing room will hold one-half day's production.
3. Each room will be wide enough to conveniently turn and manipulate a lift truck or to place three rows of products racks or cars. A 12 ft. width is adopted.
4. Each room will be 35 ft. long and 12 ft. wide.
5. The height of the roof will be a minimum in which a man can work without injury to himself. Adopt wall height of 6 ft.
6. Ridge of roof at center and parallel to side walls. Slope of roof a minimum to prevent dripping. Adopt a slope 1 ft. vertical to 3 ft. horizontal.
7. Construction shall be:
    1. Concrete block or other masonry wall 8 in. thick.
    2. Substantial insulated roof.
    3. Concrete floor 5 in. thick on 6 in. of compacted cinder fill.
    4. Doors to fit tight and be made of double sheathing with insulating material between.

ASSUMED OPERATING CONDITIONS.

1. Relative humidity of air inside of curing rooms shall be not less than 80 per cent and as near 100 per cent as it is possible to obtain.

2. Curing room and product in it shall be brought to 125° F. and maintained at that temperature.

3. In three of the curing rooms the temperature of 125° F. should be maintained simultaneously while the fourth room is being brought to 125° F. and the fifth is being emptied and filled with products. This permits of 48 hr. curing which is desirable when the outside air is above 32° F., but still cool.

4. Assume the air outside to be 32° F.

5. Assume the air in the plant building to be 70° F.

6. Assume the temperature of the block before curing to be 65° F.

7. Assume the conductivity of the curing room structure in B. T. U.'s transferred per 1° F. difference in temperature per hour per sqare foot to be as follows:

Walls = 0.30
Roof = 0.45
Ends = 0.60
Floor = 0.10

8. Assume that the product under above conditions will be brought to the 125° F. temperature in 6 hours.

Solution of heating problem is as follows:

2. Heat required to raise product from 65° F. to 125° F.

Heat losses:

B. T. U. loss in walls = 0.30 x 2 x 35 x 6.0 x 93 = 11,718 B. T. U. per hr.
" " " roof = 0.45 x 5 x 12.6 x 35 x 93 = 92,278 " " "
" " " outside ends = 0.60x60x6.7x93 = 22,431 " " "
" " " inside ends = 0.60x60x6.7x55 = 13,266 " " "
" " " floor = 0.10 x 60 x 35 x 93 = 19,530 " " "

Total =159,223

2. Heat required to raise product from 65° to 125° F.

500 block per curing room. Each room to be raised separately from 65° to 125°, or a temperature rise of 60° F.

Weight of concrete = 500 x 50 lb. = 25,000 lb.
Assume racking equipment will be 10% of block weight = 2,500 lb.

Total weight of product to be raised = 27,500 lb.

Assume specific heat of soft concrete as 0.25.

Total B. T. U. required = 27,500 x 0.25 x 60 = 412,500 B. T. U.

A concrete block 8x8x16 in. with two cores 4x5 in. in section, setting on a pallet with have exposed to the air 760 sq. in. of surface or approximately 5.28 sq. ft. Therefore, the total square feet of exposed surface of concrete in the room = 5.28 x 500 = 2,640 sq. ft.
Add 10% for racking equipment = 260 sq. ft.

Total = 2,900 sq. ft.

It is assumed the wet concrete will absorb only 1½ B. T. U. per hour per square foot of surface per degree difference in temperature.

Also assume that in bringing the product from 65° to 125° the air reaches 125° in two hours, while it takes the product 6 hours to reach 125°. This means a mean difference in temperature of 20° between the block and the air in the room while both are being raised from 65° to 125°.

20 x 1.5 x 2900 = 87,000 B. T. U. maximum that can be absorbed per hour by the product and racking equipment.

$$\text{B. T. U. required per hour} = \frac{412{,}500}{6} = 68{,}750 \text{ B. T. U.}$$

Inasmuch as the block can absorb under these assumptions 87,000 B. T. U. per hour, it is evident that the block will be able to absorb 68,750 B. T. U. which is given to it each hour.

To raise the air in one curing room requires:

Volume of room = 6.7 x 12 x 35 =	2,814 cu. ft.
Volume of block and racking equipment =	181 cu. ft.
Volume of air =	2,633 cu. ft.
Weight of air = 2,633 x 0.0684 =	180 lb.
Specific heat of air =	0.25

B. T. U. required to raise air from 65° to 125° = 180 x 0.25 x 60 = 2700 B. T. U.

B. T. U. required per hour for heating air = 2700 ÷ 2 = 1350 B.T.U.

SUMMATION OF MAXIMUM B. T. U. REQUIRED.

Heat losses from rooms	=	159,223	B. T. U.	per	hour
Heat required to raise products from 65° to 125°	=	68,750	"	"	"
Heat required to raise air	=	1,350	"	"	"
Total	=	229,323	B. T. U.	per	hour
Use		230,000	"	"	"

CALCULATION FOR BOILER.

B. T. U. in steam at pressure of 10 lb. (25 lb. absolute pressure) = 1160 B. T. U. per lb. of water.

$$\frac{230{,}000}{1160} = 198 \text{ lb. of steam required per hour.}$$

1 boiler h. p. is 34½ lb. water evaporated from and at 212° F.

On a practical basis use 30 lb. of water.

198 ÷ 30 = 6.6 boiler h. p. required.

Use 15 h. p. boiler.

Steam for power, heat for buildings, and for other purposes, is not included in this steam curing problem, but should be given consideration in the design of a complete products plant.

CALCULATION FOR AMOUNT OF COAL REQUIRED.

Assuming a boiler efficiency of 25%, B. T. U. required per hour = 4 x 230,000 = 920,000 B. T. U. per hour.

As each pound of coal has a theoretical B. T. U. of 11,000, coal requires per hour $= \frac{920,000}{11,000} = 84$ lb.

SATURATION OF AIR.

9.5 lb. of moisture is required to saturate 100 lb. of air at 125° F.

Weight of air in one curing room =180 lb.

$\frac{180 \times 9.5}{100} = 71.1$ lb. of water required to saturate the air in a curing room at 125° F.

As 159,223 B. T. U. are required per hour for heat losses in five rooms, we can assume 1-5 or 32,000 B. T. U. per room. This figure will be used in determining whether the air in a chamber will always be saturated.

32,000 ÷ 1160 = 27.6 lb. of steam.

As 17.1 lb. of steam will saturate the air, the equipment will be ample.

RADIATION SYSTEM.

Assume 2 B. T. U. per sq. ft. of pipe per degree difference in temperature per hour.

Heat loss in outside wall:
6 x 35 x 0.3 x 93 = 5,859.0.
Heat loss in inside wall when assuming the temperature in the empty chamber alongside will be 50°.
6 x 35 x 0.3 x 75 = 4,725
Heat loss outside end wall:
6.7 x 12 x .6 x 93 = 4,486.0
Heat loss inside end wall (plant at 70°)
6.7 x 12 x .6 x 55 = 2,653.0
Heat loss floor (ground at 32°)
35 x 12 x .1 x 93 = 3,906.0
Heat loss roof:
12.6 x 35 x .45 x 93 = 18,456.

Sum = 29,500 B. T. U.

One square foot of radiation surface will radiate 2 B. T. U. per hour per degree difference in temperature.

$$\text{Radiation required for outside wall} = 5{,}859 + \frac{29{,}500}{2} = 20{,}610.$$

$$\text{Radiation surface for outside wall} = \frac{20{,}610}{2 \text{ x } 93} = 111 \text{ sq. ft. of pipe.}$$

$$\text{Radiation for inside wall} = 4{,}725 + \frac{29{,}500}{2} = 19{,}476.$$

$$\text{Radiation surface for inside wall} = \frac{19{,}476}{2 \text{ x } 93} = 105 \text{ sq. ft. of pipe.}$$

Use 1¼-in. pipe.

Each lineal foot of a 1¼-in. pipe = ⅓ sq. ft. radiation surface.

Therefore, outside walls require 111 x 3 = 333 lin. ft. of pipe, or ten lengths of pipe 35 ft. long.

Each inside wall requires 105 x 3 = 315 lin. ft. of pipe, or nine lengths.

A 4-in. main line should be used from the boiler and 2-in. pipe from the main line to each room. The pipe lines should be designed with check valves so that any one of the rooms can be shut off or be in operation without affecting the other rooms. In order to provide the necessary saturation, a 1¼-in. pipe should be placed near the floor on the inner side of wall parallel with the length of the room running into a dead end. This pipe should be equipped with a spray opening or steam gauge and air cock at intervals of 3 or 4 ft. With this system, the lines of pipe should be so arranged that the piping along the outside ends will be higher than along the inside ends. This will cause the condensed steam to flow back to the main line and into the boiler.

As radiation boilers are not usually sold on a horse power basis, it is necessary to figure 10 sq. ft. of heating surface per h. p. In other words, where we require a 15 h. p. boiler, we would require a radiation boiler having 150 sq. ft. of heating surface. The smallest radiation boiler sold by a large boiler company has a heating surface of 206 sq. ft. and will supply 1600 sq. ft. of radiating surface.

In the problem above worked out, the radiating surface required is only 1,062 sq. ft.

MARKETING.

Having determined upon the plant layout and design of curing rooms, the problem of marketing can be studied. To concentrate on the problem concrete blocks or tile are discussed. There are a multitude of designs of block and tile, including every conceivable shape, division of air space and interlocking features. Of this assortment there are probably four or five designs that are good and practicable in all respects. Elimination of the poor designs can be made by inspection and a final selection arrived at by investigation and tests. No doubt the information gained would result in further improvement. Such a study should, of course, be made on a basis of:

1. Building loads.
2. Fire resistance.
3. Conductivity.
4. Competition with other building materials.
5. Salability.
6. Acceptance by building trades.
7. Durability.
8. Appearance.
9. Economy in manufacturing cost.
10. Ease in handling and laying up on the job.
11. Strength to prevent breakage in handling.
12. Strength in wall.

With respect to consistency, there are two generally known methods of products manufacture—the dry and the wet. It is generally agreed that the dry consistency is below the consistency giving maximum strength, and the wet is above the consistency giving maximum strength. In the curing of the product some method or other may have considerable bearing on quality cost results. An investigation including these factors would no doubt enable us to arrive at the comparative economy of these methods.

The effect of the method of curing on products is an important subject on which there is little actually known. The consistency of the tamped or pressed concrete product is too dry. The questions that arise are: is a warmed saturated air beneficial and is it an economical manufacturing operation? What temperature should prevail in the curing chamber for most economical production? To what extent are products improved or harmed by alternate wetting and drying, such as in sprinkling? What happens when products are subjected to various weather exposures and what practice should be followed?

It is generally agreed that with a continuous mixer one man's labor is saved in comparison to a batch mixer, but is a continuous mixer dependable? Will it turn out as good concrete constantly as a batch mixer, and is it as economical?

In a products plant where comparatively dry consistencies are used and where in molding, maximum plasticity and uniform mixing is desirable, is it practicable to mix the concrete longer than one minute? It is hoped that these questions will be answered before the next convention of the Institute.

More exact data are needed on the performance of products making machinery from the standpoint of production, quality, uniformity of product and mechanical troubles. Interesting data could be secured from products manufacturers who might co-operate to the extent of keeping a record along lines suggested by the Committee. Such a record, of course, should be kept over a considerable period of time to be of real value.

PLANT OPERATION PRACTICE.

1. It is desirable to use materials which will produce the quality desired at the least cost for those materials. The cheapest priced aggregate sometimes requires more cement and, therefore, results in a higher cost of the total materials than some other aggregate. Sometimes the choice of an inferior aggregate results in excessive breakage of product in manufacturing, thereby adding to its cost.

2. Aggregates should be selected with the best possible grading. Generally, the maximum sized aggregate should have a diameter of one-half the thickness of the thinnest wall of the product. As much coarse aggregate as possible, that is, those sizes larger than sand, should be used. Such aggregates do not require as much cement for equal strength concrete.

3. The water content of the concrete should be carefully established and kept constant. Tamped or pressed products should be made of concrete having as much water as can be used. In wet cast products, as little water as possible should be used.

4. After molding the product, it should be cured under conditions which will produce a uniform, satisfactory quality by the time the product is delivered to the purchaser, or at 28 days. Products should be kept at temperatures above 80° for at least 24 hours before placing them on the outside, where temperatures should not fall below 60°.

Products that are to be exposed to temperatures below 60° and above freezing, should be kept in a temperature above 80° for not less than 48 hours. Where the outside temperature is expected to drop below 32° within 7 days, a longer period of curing should be used. It is believed that higher curing temperatures, say up to 125° F., produce better and quicker results. In hot, dry weather, products should be kept damp by sprinkling for about 7 days after they are taken from the curing rooms.

Products should be handled no more than absolutely necessary for the first 24 hours after they are molded. When they are handled, and especially directly after being taken from the machine, jarring is very apt to cause damage, especially to thin products that do not lay flat.

It is necessary, however, to remove products from the machine to a place for curing. This is usually accomplished by hand, carts, suspended conveyors, track and car system or trucks. The lift truck scheme is new and therefore, naturally, there are those who object to it. It is believed that where it has been tried under proper conditions, it has proved successful and economical. It is advisable, however, to have a smooth floor kept reasonably clean. The truck should be of the type which lowers slowly and easily, and it should be equipped with rubber tires and have springs suitable for the load to be carried. Lift, narrow gauged cars are now being produced which eliminate the necessity of keeping the car itself in the curing chamber. In this manner, the life of the car is increased considerably.

S. H. WIGHTMAN, *Chairman.*

REPORT OF COMMITTEE P-7, ON CONCRETE PIPE, DRAIN TILE AND CONDUITS.

Prior to 1921 the Institute specifications for concrete pipe were included in the general specifications for monolithic sewers, the whole subject being in charge of the committee on sewers. This committee in its annual report to the Institute in 1921 recommended that a separate specification be written for concrete pipe and that this specification be issued independent of the existing specification for monolithic concrete sewers. In compliance with this recommendation, and in line with the general policy of the Institute a new committee known as Committee P-7, on Concrete Pipe, Drain Tile and Conduits was named to take over the question of concrete pipe and kindred subjects. Due to delay in the formal acceptances by appointees on this committee, the committee could not proceed with its work until sometime had elapsed after the last convention of the Institute, and as members of the committee are located at points so widely separated as the cities of New York and Los Angeles, it became immediately apparent that formal meetings of the committee would be an impossibility and that the work of the committee would, of necessity, be conducted by correspondence. This method is now being carried on by the submission to each member, through the secretary of the committee, of a series of fundamental questions relating to the committee's work. Answers to the questions submitted are compiled by the secretary and again forwarded to each member.

Work carried on in this manner, while it is thorough, is of necessity slow and it entails upon the secretary of the committee a task that is unusually difficult and arduous.

The first work of the committee was to act in conjunction with Committee S-3, on Monolithic Sewers, to eliminate from the present Institute specifications on concrete sewers all reference to the subject of concrete pipe. This was successfully accomplished.

The next step undertaken was to effect a more complete sub-division of the work assigned to the committee. It is the opinion of the committee that where a distinct use of pipe necessitates a separate design, such pipe shall be recognized as a sub-division of the general problems of pipe and shall have a separate specification. The following sub-division is recommended:

(a), Plain concrete pipe.
(b), Reinforced-concrete pipe.
(c), Concrete pressure pipe.
(d), Concrete irrigation pipe.
(e), Concrete drain tile.
(f), Concrete conduits.

Your committee has decided, for at the present at least, that it will undertake no work on the subjects of pressure pipe, irrigation pipe and concrete conduits.

Your committee is of the opinion that it is preferable, where existing specifications issued by recognized authorities, meet with the approval of the Institute that the Institute accept and adopt such specifications rather than issue additional standards. Your committee after a careful examination of existing standards and pending final arrangements for their publication by the Institute, recommend the following:

The adoption by the Institute of Standard Specification C-14-20 of the Amerian Society for Testing Materials for Cement Concrete Sewer Pipe and also the adoption by the Institute of such parts of the Standard Specification C 4-21 of the American Society for Testing Materials as relates to the manufacture of concrete tile.

In relation to reinforced-concrete pipe, your committee recognizes in principle the value of classified standards for this commodity; the classes to be of as small a number as will economically cover the demand for this type of pipe. With this principle in mind, your committee has now under discussion the fundamentals in relation to the problem of reinforced-concrete pipe. Your committee will bear in mind the fact that the Institute is represented on the Joint Concrete Culvert Pipe Committee and will endeavor to conform to the action of that Committee as it affects pipe for use in culverts.

The committee hopes, within the present year, to complete its work in the preparation of a tentative specification for reinforced-concrete pipe and to begin work on some of the other subjects that have been so far postponed.

ERNEST F. BENT,
PAUL KIRCHER,
WILLIS WHITED,
C. H. BULLEN,
R. S. LANDER,
G. A. HOOL,
M. L. ENGER,
B. S. PEASE,
JOS. S. LAMBIE,
A. M. HIRSCH,
HOWARD SCHURMAN.

H. W. LOVING, *Secretary.*
C. F. BUENTE, *Chairman.*

REPORT OF COMMITTEE C-3, ON TREATMENT OF CONCRETE SURFACES.

Last year the report of your committee was mainly devoted to a description of the methods used in the treatment of exteriors of industrial reinforced-concrete buildings. It was the intention of the committee this year to present this matter in the form of a Standard Recommended Practice, but pending the report of the Committee on Form of Standards, it seemed desirable to postpone this presentation until next year. On this account our Sub-Committee II, under the chairmanship of Mr. Horr, offers no report in connection with the report of the general committee.

Sub-Committee I, on Portland Cement Stucco, suggests some additional revisions to the standard recommended practice as adopted in 1920.

Sub-Committee III, on Special Decorative Treatments, makes a beginning this year and offers a brief progress report.

The major portion of the present report deals with the treatment of interior concrete surfaces, a subject which has brought to light numerous differences of opinion, particularly in regard to plastering concrete surfaces and the use of waterproofings. In fact the committee has approved this report with reservations and the subject matter is presented mainly for the purpose of inviting constructive criticism, and, of course, for the information of those who may make use of it. Mr. Ahlers and the members of this sub-committee are to be commended for offering definite recommendations in regard to practice which is at present very far from being standardized, and regarding which many conflicting opinions are held.

Sub-Committee V, on Bond of Applied Coatings, presents the results of some interesting preliminary experiments on the adhesion of stucco coats.

SUB-COMMITTEE I—PORTLAND CEMENT STUCCO.

In addition to changes in the Recommended Practice for Portland Cement Stucco recommended by the Sub-Committee in 1921, the committee recommends for adoption the following changes:

Article 2, Paragraph 1:

Insert before the final sentence, the following sentence. "The end joints should be thoroughly protected with sheet lead flashings."

Add the following sentence at the end of the present paragraph: "Double flashing should be used at the intersection of sloping roof with metal lath wall. The metal lath should be stapled into the upper or cap flashing at its upper edge which is rigidly attached to the wall behind."

In the notes on the page opposite add the following paragraph after the first: "Where horizontal wood trimming courses occur in stuccoed walls satisfactory results have been secured by making these wood courses of stock approximately 2 in. thick, rabbetted on top with the stucco extending down over the inside lip, thus doing away with flashing. Where flashings are used on furred walls, it is important that they extend back of the furring strips."

Article 4:

Add the following clause at the end of the fifth sentence: "Nor should any wooden members (except inserts for nailing) be embedded in masonry walls, particularly where they are to be covered with stucco."

Article 9, Paragraph 2:

Insert "outside" before "face" in third line of first sentence.

Also make same changes in notes on page 5 under "Frame Walls," second line of fifth sentence.

The word outside is added in order to make clear which face of the studs is meant. The point has been raised that the horizontal brace should be flush with the inside face of the back plastering so as to act as a fire stop, but with fire stops at floor and ceiling as recommended in the second paragraph of the notes on p. 5, it seems unnecessary to make the horizontal bracing also act as a fire stop, especially since the end sought is difficult to accomplish in practice without breaking the continuity of the stucco at that point.

Article 15, Paragraph 2:

Change "6-in." to read "10-in."

Add to the notes referring to this paragraph the following: "The reinforcement obtained by the use of metal lath around corners over wood lath is important on both interior and exterior angles."

J. E. FREEMAN, *Chairman,*
Sub-Committee I.

SUB-COMMITTEE III—SPECIAL DECORATIVE TREATMENTS.

For the purpose of encouraging the wider use of concrete for architectural purposes, it is the present aim of the committee to compile and describe the methods of obtaining ornamental finishes and decorative effects as used by specialists in concrete work. It is also the intention of the committee to procure photographs of finished structures, and from time to time to publish those which best illustrate the range of possibilities in this type of ornamentation.

Method Used by Thompson & Binger on Hide & Leather Building, New York City.

The concrete was made with the ordinary grey portland cement and regulation aggregates. The facing mixture was composed of white portland cement and colored aggregate, in this instance consisting of rough quartz and feldspar chips mixed one part cement and two parts aggregate.

In placing this mixture metal lath was wired to the outer reinforcing bars, leaving a space of 1½ to 2 in. between it and the outer form. This space, usually filled with ordinary concrete as fireproofing, was used for the facing mixture. By keeping the level of the facing concrete higher than the backing concrete, no difficulty was experienced in securing a continuous face showing none of the backing. Careful tamping of the facing mixture was accomplished by ½ in. bars with T-heads. After the forms were removed any slight pointing up of defects in the surface was done and the concrete allowed to cure all through the winter before tooling was done. This consisted of the regular method of bush-hammering to expose the aggregate, and the resulting surface is almost an exact reproduction of natural granite. This was done with the lower stories and the upper stories were surfaced by grinding with carborundum. Coloring with Minwax waterproofing of a natural concrete shade brought the surface to an even pleasing color. This building is a striking example of what can be done in surfacing reinforced concrete.

Methods Used by Pennsylvania Railroad.

Finishes Required: All showing faces of concrete work shall be given one of the following special finishes:

(a) Scrubbed finish with granolithic facing.
(b) Scrubbed finish without granolithic facing.
(c) Tooled finish with granolithic facing.
(d) Tooled finish without granolithic facing.

The underside of concrete bridges over railroads and streams shall be finished by the filling of any voids which appear, and brush-coating with cement to a uniform appearance.

The underside of concrete bridges or bridge decks over city streets shall have the treatment just described unless some further treatment is called for on the plans or in the specifications.

Granolithic Facing: Granolithic facing shall be composed of one part cement, 1½ parts coarse sand or gravel, and 2½ parts coarse aggregate, made into a stiff mortar.

The sand shall contain no pebbles that will be retained on a ¼ in. sieve. Enough sand for the entire face shall be set aside at the outset to insure uniformity of color.

The coarse aggregate shall be granolithic grit, or pebbles, as specified. Grit shall be clean crushed granite or trap rock. Pebbles shall be washed and screened. Samples of grit or pebbles weighing not less than three pounds, shall be submitted for approval before ordering for the work; and enough material for facing the entire work shall be delivered at the outset to insure uniformity in the face work.

The grit or pebbles shall pass a ⅜ in. sieve and be retained on a ¼ in. sieve. If the material as delivered on the work contains larger or smaller sizes, they shall be screened out before using, and sizes rejected may be used in other portions of the work, but not in the face work.

FINISH C—PENNA. SYSTEM—NORTH PHILADELPHIA, PA.

Application: For vertical surfaces the facing mixture shall be deposited against the face forms to a least thickness of 4 in., kept separate by a float board on edge, by skilled workmen, as the placing of the concrete proceeds, and thus form a part of the body of the work. For bridge seats and other horizontal courses the mixture shall be deposited on the concrete to a least thickness of 1½ in. immediately after the concrete has been tamped and before it has set, and shall be troweled to an even surface. Thin sections, such as railings, parapets, etc., and other portions where facing is not practicable, will be made throughout of the facing mixture, and the contractor shall so estimate. The embedding of stones of the largest practicable size in such portions, is permissible so long as no stone is nearer to the surfaces than 3 in. Care must be taken to prevent the occurrence of air spaces or voids in the surface, or any separation between facing mixture and the body of the work. The facing mixture must be so spaded as to force the coarse aggregate against the forms and into all corners and edges. Films of scum or grout must not be allowed to lie o nthe facing mixture, and each layer as added must be well chopped into that below, so as to avoid the appearance of lines or streaks.

Scrubbed Finish: For scrubbed finish the contractor shall have available at least a ½ in. hose stream of fresh water, continual flow. The face forms shall be removed as soon as the concrete has sufficiently hardened, and any voids that may appear shall be filled with the mixture.

The surface shall then be immediately scrubbed with brushes and water until the coarse aggregate is exposed, and then rinsed clean and protected from the sun and kept moist for three days.

Minor defects may be patched, but all patching must be done as soon as practicable, and before the body of the concrete has dried out. Serious defects will require the work to be replaced, and for face work, containing considerable detail work, mouldings, etc., the forms shall be painted as an additional precaution against injury to the surface in removing them.

After scrubbing and rinsing a portion of the face, all other portions shall be washed down vigorously with the hose stream to remove the cement water which has flowed over them.

After pouring a section of face work, the entire face below shall be washed down vigorously with the hose stream to remove all traces of grout which may have leaked or splashed onto them.

When the entire work is finished and otherwise ready for acceptance, the face shall receive a complete and thorough washing and scrubbing; and acid shall be used at this time if directed, to remove stains, and thoroughly rinsed off.

Tooled Finish: For tooled finish the forms shall remain in place at least three days, and for ornamental work longer, as the Engineer may direct. The workmanship on the forms shall be in no way inferior to that for scrubbed finish, and on removal shall leave perfect edges and an absence of joint marks, bulges, etc.

All faces shall be tool dressed to remove the cement film and expose the coarse aggregate after the lapse of at least one month. Generally the tool shall be a six-cut "patent hammer" or "bush hammer," and for small details, molding, edges, etc., the dressing shall be done with the tool best suited to the purpose. This detail work shall be done by journey-men stone cutters, or workman having equivalent experience and capable from the outset of securing perfect edges and finish acceptable to the engineer.

E. D. BOYER, *Chairman,*
Sub-Committee III.

SUB-COMMITTEE IV—INTERIOR OF BUILDINGS.

The treatment of interior concrete surfaces is, in a broad way, similar to that of exteriors except that interiors are not exposed to action of the elements and usually it is desirable to produce smoother finishes and finer details.

To get a better understanding of the sub-divisions covering this subject the report of Sub-Committee II on Exterior Finish as printed in *Proceedings*, American Concrete Institute, 1921, pp. 308-14, should be read first, as reference to this report are made herein.

The important points of difference in treatment of exterior and interior surfaces are brought about by the action of water and frost. On exteriors these agencies get at the entire exposed surface while on interiors frost does not usually occur, and the problem of stopping the penetration of moisture or dampness through the walls to the back of the finish, or of stopping condensation on the surface, is the most difficult one to solve.

The treatment of interior concrete surfaces can be separated into the following sub-divisions.

I. Pointing and smoothing.
II. Waterproofing.
III. Cement Wash.
IV. Painting.
V. Plastering.

I. Pointing and Smoothing.

To this applies all that has been said under the general Committee's Report in the Proceedings of the Institute for 1921, p. 309 (Sub-Committee II, Method 2, Cleaning and Patching.).

A finer sand may be used for pointing interiors; also the pointing mortar may contain an addition of hydrated lime up to 20% by volume of the cement.

If a very smooth surface is required a power grinder of commercial make may be used, if the area is large, say 15,000 sq. ft. or over. With an investment of $300 for a small twenty pound machine the work can be done for 2 cents to 4 cents per square foot. By using long handled carborundum stones the work can be done by hand labor ($5.00 for 8 hours) at a cost of about 6 cents to 8 cents per square foot. The cost will vary with smoothness required and with the age of the concrete and toughness of the aggregate, but here as in exterior work better results are obtained by doing the rubbing as soon as possible after the concrete is set.

Under no condition should a cement wash be brushed on a concrete surface if it is expected to paint the surface later.

It is desirable to wire brush the entire surface in all cases except where a cement wash is to be applied, and even for a wash wire brushing is not objectionable especially on all parts where mortar and water from concrete have run down the surface.

II. Waterproofing.

A. Under Hydrostatic Pressure.—In most cases where considerable hydrostatic pressure is encountered on new construction, the waterproofing treatment, usually of the membrane type, will be applied to the outside of a wall that has been placed with due regard to proper proportioning, mixing, and depositing of the concrete so as to obtain a compact mass free from seams or day's work planes. Under low heads the membrane treatment may often be omitted.

In cases where minor leaks occur at seams or joints these must first be stopped by cutting back the joints and sealing with a cement mortar to which some accelerator such as calcium chloride has been added to hasten hardening (3% by weight of the cement is effective). In cases of considerable seepage temporary bleeder pipes may be necessary to relieve the pressure until the mortar has hardened, after which the bleeders may be removed and the holes similarly sealed.

The entire surface of the wall may then be plastered with two or three coas of cement mortar thoroughly mixed and carefully applied by hand. Where a machine such as the cement gun is used one coat may be sufficient. If desired as an extra precaution an integral waterproofing compound may be used in the mortar provided it is durable and will not materially weaken the strength of the mortar.

Wall waterproofing treatment under hydrostatic pressure should preferably be done by those specializing in such work who can furnish bond and guarantee of satisfactory results.

B. Dampproofing Walls.—In buildings where condensation of moisture on interior surfaces of exterior walls above ground is likely to occur or where definite assurance against dampness is desired, the walls should be insulated by means of furring and lathing or furring tile before plastering, unless such insulation has been provided within the wall itself as in the case of double wall construction.

In buildings where assurance against dampness is of secondary importance a single coat of cement plaster (to which 20% of hydrated lime by volume of cement is added) may be applied on the well cleaned and roughened surface, and the surface should be left in condition to receive either finish plaster or paint as may be called for.

In the majority of cases the problem is not so much one of checking penetration of moisture through the wall as of sufficient insulation to prevent condensation of moisture on the interior wall surface.

III. Cement Wash or Rubbed Finish.

See 1921 *Proceedings*, A. C. I., Committee on Treatment Concrete Surfaces, pp. 311-12, Sub-Committee II, Method 3, Cement Wash, and Method 4, Rubbed Finish.

The same recommendations apply to the treatment of interior concrete surface as to exterior treatments.

IV. Painting of Cement and Concrete Surfaces.

A. With Linseed Oil Paints.

Preparation of Surface: If the surface is freshly made or contains any substantial percentage of free lime (oxide or hydrate), there should first be applied a neutralizing liquid made by dissolving 3 lb. of zinc sulphate crystals in one gallon of water. This should be applied by a brush, spray, or similar means. It will react with the free lime, forming neutral pigments (calcium sulphate and zinc hydrate) which will have no effect upon subsequently applied oil liquids. After allowing a period of twenty-four hours for the thorough drying of the cement or concrete surface, the paint may be applied by brush or by spray.

Painting Interior Surfaces of Cement and Concrete: Any linseed oil paint desired may be applied to interior concrete surfaces. It is suggested, however, that for white and light tints, paints in conformance with specifications of the U. S. Interdepartmental Committee (Circular 89 of the Bureau of Standards) will give excellent results. Where a flat or matte surface paint is desired, the use of flat wall lithophone paints of the type recommended in Circular *111* of the Bureau of Standards is advocated. Mill whites of the gloss, egg-shell or flat types, made of lithophone and zinc oxide combinations, are suggested where high illumination is desired in factories.

B. Water Paints.

Paints of either the hot or cold water type may be applied to almost all concrete surfaces. Rough cast concrete left as it comes from the forms may successfully be brushed, painted, or sprayed. In most cases, however, it is better for appearances to point and smooth the surface as described under sub-division I of this report, inasmuch as air holes and rough spots will be sure to cast heavy shadows after the coating has been applied and therefore be more apparent than would be the case if no coating had been applied to the surface.

An excess of form oil or grease on the surface of the concrete will be injurious in two ways to this coating—first by preventing proper bond between the paint film and the concrete, resulting in scaling or peeling; second by possibly penetrating the paint and causing unsightly stains to appear on the surface.

If forms are greased where the surface is later to be covered with water paints, care should be taken that this coat is as uniform and thin as possible, and it may be desirable to use soft soap as a form coating where water paints are to be applied.

The same smoothness of surface will be apparent before and after applying the water paints and the amount of smoothness should therefore be determined upon and obtained before the cold water paint is applied, as the water paint does not obliterate roughness, board marks, or texture but merely gives the surface a uniform color.

The porosity of the surface will determine the amount of paint to be used on a given area as the rapid absorption of the liquid from the paint has the effect of thickening the paint film, though this may be overcome by first dampening the wall with a light spray of water or by diluting the paint to a consistency thinner than is desirable for less absorbent surfaces.

There is a very wide difference in the covering capacity and in the hiding power of the various water paints on the market.

The user of cold water paint should consider the covering capacity as compared with the price, inasmuch as the more expensive material may often be less expensive per square foot area of finished work.

Where prepared compounds are not available or not desired a coating of whitewash may answer the purpose as well as the more expensive patented products. In such cases the addition of a binder will create a more durable coating, and it is possible to prevent powdering and scaling in this manner.

A Government light-house formula for whitewash will work well on concrete. The adhesives to be recommended are flour paste or glue. Linseed oil may be added as an adhesive. It should then be mixed with the lime and water while still hot from the slaking of the lime.

The following formulas may be used, and will be found not to rub off excessively after application?

(a)—Slake 50 lb. white lump lime to paste. While hot stir in one gallon boiled linseed oil. Thin to consistency of cream with water. If yellow or cream color, add laundry bluing dissolved in water.

(b)—Slake 100 lb. white lump lime, strain, add 5 gallon strong salt brine, 5 gallons cooked flour paste, 1½ gallons dissolved glue. Thin with water to working consistency.

V. *Plastering.*

1. *Furring.**—If it is desired to plaster the interior surface of an (See Section II, B, Dampproofing Walls, page —). If furring is not used there is a possibility that the penetration of moisture through the wall may injure the decorations on the plaster. There is also the possibility that the relatively high heat conductivity of the wall will cause condensation on the interior surface, thereby injuring the decorations and will also be uneconomical in the amount of fuel required to heat the building. The use of furring will very largely overcome these difficulties.

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*It is the committee's intention to prepare further recommendations regarding the use of furring and bituminous bond coats during the coming year.
exterior concrete wall it is recommended in general that furring be used.

2. Where plaster is to be applied directly to concrete the following requirements should be included in the specifications:

3. -*Roughness.*—The surface of the concrete should be rough. If it is known beforehand that the concrete is to be plastered the desired degree of roughness can be obtained by the use of unplaned lumber for forms or by some similar device. If the concrete is quite smooth, it must be roughened and this can be best accomplished by chopping it with a hatchet. The roughness is very necessary in order that a proper bond between the plaster and the concrete can be obtained. A power surfacer is in the market as mentioned under Sub-division I, provided with a disc on which are mounted a number of steel cutting wheels that chip the surface of concrete and leave it roughened in much the same manner as a bush hammer would do.

4. *The concrete should be clean.*—This means that it must be free from loose particles, from dust, from laitance, and from grease. Dust, loose particles, and laitance will be sufficiently removed by the roughening process specified in the preceding paragraph. If grease spots occur over considerable areas they should be burnt off with a blow torch.

5. *The concrete should be wet.*—If it is possible to apply the plaster before the green concrete has completely dried out, then no wetting is required. If, however, the concrete has dried, then it must be wetted before plastering. The degree of wetness is extremely important. If the concrete is too dry, it will suck water out of the plaster rapidly, which may increase the difficulty of spreading the plaster and which will certainly impair the final hardening of it. If the concrete is too wet, the surface pores will be so filled with water that it will be physically impossible to push the plaster into these pores. This will result in a very weak bond between the plaster and the concrete, or in extreme cases it may be found impossible to apply the plaster. If the concrete it wetted by the application of water to its surface, as it must be in all cases except where the plaster is being applied to green concrete, then the appearance of the surface is not always an indication of the wetness of the concrete. It takes some time for the mass of dry concrete behind the surface to suck in the water. At present it requires considerable practical experience in order to determine just what is the proper degree of wetting.

6. *Thickness of grounds.*—The principal consideration in determining the thickness of grounds in plastering on concrete is that the plaster should be as thin as possible. Besides a saving of material the thinness of the plaster coat will tend to prevent cracking and will lessen the weight which the concrete has to carry. Because of the fact that the concrete is rough and that this roughness is necessary to obtain a good mechanical bond, there is a minimum thickness of plaster below which it is impractical to go. The thickness which is usually specified is ⅝ in. It is recommended that this be reduced to at least ½ in. or ⅜ in. if possible.

7. *Number of coats.*—Where it is desired to have a smooth troweled finish it will be necessary to apply the plaster in two coats. The first coat should be bonded thoroughly to the concrete by troweling it into all of the depressions and pores. It should be brought out flush with the grounds and darbied to a true and even surface. The second coat should be as thin as it is possible to make it without having the first coat show through. This should be troweled down to a smooth, hard finish. Where a smooth, troweled finish is not required one coat will be sufficient but this can not be a white coat. If this has been brought out to grounds and darbied true it may be further finished by means of a wood float or in any other approved manner. A single coat of white plaster is almost certain to eventually loosen from a concrete surface.

8. *Materials.*—For the first coat, it is recommended that a mixture of one part of hydrated lime to three parts of sand, by volume, be used. This may be masons' hydrate or finishing hydrate. For the second coat of a smooth, troweled finish, use two volumes of lime putty made by soaking finishing hydrate with an excess of water for at least 48 hours before using, and adding one volume of dry calcined gypsum immediately before application.

J. G. AHLERS, *Chairman,*
Sub-Committee IV.

SUB-COMMITTEE V—BOND OF APPLIED COATINGS.

For the preliminary tests for measuring the bond of applied coatings by freezing and thawing specimens of two coat stucco work on a hollow tile base were used. Three 8x12x6 in. tile, one soft, one medium and one hard burned were selected and then split lengthwise to give six specimens 8x12 in. in size. The plaster was hand mixed in the proportions 1 : 3 by weight to plastering consistency.

Before applying the first coat grounds were fastened to the sides and ends of the base to obtain a coat exactly ⅜ in. thick. The face of the tile was thoroughly dampened and just before applying the plaster it was wiped with a piece of waste so no free water was left on the surface. The coat was applied with a trowel and screeded to an even surface and as intial set was taking place floated with a wooden float. The grounds were removed three hours after coating and the specimens placed in a damp closet for three days, then exposed to the air of the laboratory for four days before applying the second coat.

In order to bring in some variation, three treatments of preparing the first coat to receive the second were introduced. The coats of the hard burned tile specimens were saturated with water and free water was allowed to stand on the surface when the second coat was applied. Those on the medium burned tile were thoroughly dampened with no free water. The coat of the soft burned tile specimens were treated with a thick creamy grout. This was brushed on in a thin uniform coat just before the application of the second coat. The second coat was ¼ in. thick in all cases and the method of applying was the same as that used for the first one. The specimens were again stored in the damp closet for three days and then exposed to the air of the laboratory.

The specimens were allowed to age in the open air of the laboratory for nine and one half months when one of each kind was subjected to alternate thawing and freezing action. The period of frost action varied from 7 to 16 hours and the specimens after being subjected to 167 freezings show no sign of disintegration or splitting of the coats from each other or from the base.

If the results of these freezings and thawing tests can be taken as a measure of the effect of actual weathering, stucco applied as it was applied to these test specimens would last indefinitely and there would be no need to fear separation and falling of finish coats. The only interpretation which can be placed on these results at the present time is that the experimental work was better than the average found in actual practice and the application of stucco finishes as ordinarily done on the job must be used to determine the conditions which give the best bond.

While it is not thought mechanical devices will give a true measure of the factor which we are seeking, attention is called to the work done by the Bureau of Public Roads on this subject published in the June, 1919, number of Public Roads.

FRANK A. HITCHCOCK, *Chairman*,
Sub-Committee V.

Respectfully submitted in behalf of Committee C-3,
J. C. PEARSON, *Chairman*.

DISCUSSION.

Mr. Brunnier.

H. J. BRUNNIER.—Did Mr. Pearson mention what we call hard wall white finish plaster coat, placed direct on concrete?

Mr. Pearson.

J. C. PEARSON.—I mentioned the fact that the evidence we have does not seem to warrant the recommendation of the use of wall plaster.

Mr. Brunnier.

MR. BRUNNIER.—We have a good deal of that in San Francisco. I recall particularly a knitting mill in which there are a good many machines operating, with considerable vibration. We put this coat right on on the rough concrete, and it has been on eight years now and is still there and has not cracked in a single place. There are any number of other buildings where we have done the same thing.

Mr. Pearson.

MR. PEARSON.—That was not a straight gypsum coat.

Mr. Brunnier.

MR. BRUNNIER.—No, it was what we call the hard wall finish coat; instead of putting the scratch coat on first, we put this finish coat. It is lime.

Mr. Pearson.

MR. PEARSON.—That's all right.

Mr. Ahlers.

JOHN G. AHLERS.—As a sub-committee chairman, I want to add one word to Mr. Pearson's report. As he said, considerable discussion has been developed among the members of my particular sub-committee, and I would earnestly ask all the members of the Institute to read it through and give the committee the benefit of their ideas and thoughts on the subject. There has been much difference, honest difference of opinion, among the members of the sub-committee as to the type and amount of furring wanted and the method of treating the interior of concrete buildings. I would like the interest and criticism and advice from members of the Institute during the coming year, to help the committee out in its work.

Mr. Nichols.

CHARLES E. NICHOLS.—I would like to ask the chairman of the committee, first, if the evidence which they considered indicated that there was no need of using bituminous compounds or paints on concrete surfaces ordinarily roughened to insure sufficient bonding of the plaster coat to them, and provided there were indications of need for it, whether any particular types were investigated.

Mr. Pearson.

MR. PEARSON.—The committee's feeling on that subject is that the bituminous bond coats are not to be recommended. I cannot cite the evidence very well, and the committee itself has done no investigative work, but nearly all sources of available information have been drawn upon, and it seems to be the consensus of opinion of those who have participated in the work—and that includes the plastering conference which originated at the Bureau and the report of which will soon be issued—that the best recommendations were against the use of bituminous bond coats.

Mr. Cohen.

A. B. COHEN.—In connection with the bituminous bond coat, the Lackawanna R. R. at South Orange, N. J., built a flat slab viaduct 400 ft. long, and a station underneath this viaduct. They used in the station court the bituminous paint or compound to plaster the underside of the

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Mr. Cohen. slab. That has been on at least six years, and with all the vibrations of the trains overhead, there has not been a single crack in the plaster. I think the surface was not roughened to any extent, but just the rough form lumber was used, and then this bituminous paint or bonding coat was applied to the underside of the slab. Then it was plastered with, I believe, just the one coat. There is a field, I think, for the bituminous paint or bonding plaster on the outside of a slab.

REPORT OF THE SPECIAL COMMITTEE ON THE APPLICATION OF METAL FORMS TO REINFORCED-CONCRETE CONSTRUCTION

In the progress report of the committee last year the intention to adhere to column and floor form systems for building construction was stated. The use of metal forms for chimneys, sewers, retaining walls, etc., has been an accepted practice for some time among contractors handling those types of construction, but in nearly every case the steel forms offer advantages in lightness, portability, strength, or ease of assembly that cannot be obtained with wood forms which for such work are usually heavy and cumbersome.

For building construction, however, wood forms are relatively light and easily handled. Because of their practically standard character on all buildings the labor operations of erecting and stripping can usually be performed by skilled men at very low cost. It is therefore in the application of metal forms to column and floor systems in industrial building construction of reinforced concrete that the closest study is necessary if the contractor is to obtain a satisfactory job.

On every job there are always a number of factors, sometimes conflicting, that have to be given consideration in planning the work. A satisfactory job is one in which the greatest number of these factors can be embodied to the advantage of the operation. Among the factors that will occur off-hand to almost any contractor are: Speed of work, condition of labor and lumber markets, ability and efficiency of labor, area and height of building as affecting number of uses that the forms will give, summer or winter weather, etc., etc.

It is only by considering each and every one of the numerous factors involved, by careful study and by analysis of each job as a separate problem, that a contractor can determine whether wood or steel forms should be used. No sweeping generalizations are possible; a factor of minor importance on one job may be of major importance on the next. It was the feeling of the committee that the greatest importance be given this matter of analysis of conditions.

Some of the points to be considered by a contractor before committing himself to either wood or steel forms are given below.

I. *General*

- (a) Cost of materials; labor making, erecting and stripping form centering.
- (b) Rental cost of metal form panels.
- (c) Cost of erecting and stripping metal forms.

  Note: Items 1a and 1b may become one if the erection of metal forms is let on sub-contract.

(d) Will the metal forms be delivered to the job in time to avoid delay to the construction work?

(e) Will the use of metal forms require employing a mechanical trade not otherwise needed on the building, and if so, will this possibly lead to labor complications?

(f) If metal forms are handled by sheet metal trade and carpenter labor is scarce, labor conditions may be improved by the use of metal forms.

(g) Will the use of metal forms necessitate purchasing lumber for various other purposes, such as bucks, grounds, bulkheads, etc., for which old form lumber is originally used?

(h) If metal floor forms are used, can they be utilized also for wall forms, saving the cost of wood although the cost for labor on metal forms may be high?

(i) If wood forms are used for walls, will the excessive material cost overbalance possible economies otherwise offered by metal forms?

(j) Will the smooth surfaces resulting from the use of metal forms be desirable as giving a better appearance, reducing the cost of pointing or painting or other finishing, or as giving results in any way for which the owner would be required to spend additional money if wood forms were used?

(k) Will the smooth surfaces, or the possibility of their being oil-soaked, be detrimental and require the expenditure of money for special treatment preparatory to painting, plastering or otherwise finishing the surfaces?

(l) Winter weather construction. Metal forms to a slight extent lessen the fire hazard where salamanders are in use, and because of the heat-conducting qualities of the metal make it easier to keep the poured concrete above 70° while setting.

II. *Rental Basis.* If the general contractor rents the metal form panels and erects with his own labor the following points should also be considered:

(a) Will the use of metal forms cause labor difficulties due to jurisdictional disputes as to which trade should handle them?

(b) If metal forms cannot be handled by carpenters will this disorganize the labor forces by necessitating intermittent hiring and discharging the different trades, or can the job be organized to give continuous employment to all trades, thus making employment attractive to the best workmen?

III. *Sub-Contract Basis.* If the erection and handling of metal forms is let on a sub-contract either to the form manufacturer or to an outside erector, the following points should be considered:

(a) Will the introduction of another sub-contractor tend to slow up the progress of the job and thereby result in an increased cost of any part of the work? On rush jobs this question of the general contractor losing the absolute control over the speed of the work that he has when all the form-work is done by his own organization, may be of extreme importance.

(b) Will the introduction of another sub-contractor tend to simplify or complicate the labor situation? If local labor is scarce, bringing in an outside organization may relieve the situation somewhat.

(c) Can the sub-contractor be depended upon to co-operate with the others to the best advantage of the job as a whole; or will his presence (since he will be primarily interested in the cost of his own work regardless of others) tend to increase the cost of other work?

Some of the items in the list just given may be discussed a little more fully to good advantage. There is for example the first item on the list, I (a) Cost of materials, etc. For a square bay of wood forms there will be required, for example:

..............................ft. B.M. ⅞″ roofers plus waste allowance.
..............................ft. B.M. 3″ x 4″ spreaders
..............................ft. B.M. 1″ x 4″ or 1″ x 6″ battens
..............................ft. B.M. 4″ x 6″ headers
..............................ft. B.M. 4″ x 4″ shores
Total, ...ft. B.M. required per bay of..................sq. ft.

For this lumber there will be several charges:

(a) Purchase price of material.
(b) Labor to receive (unload from truck and pile).
(c) Labor to handle from pile to bench, and labor fabricating.

The total of these three items, divided first by the number of uses the forms will give, and second by the number of square feet per day, will give what is practically the material cost per square foot. To this must be added the labor cost per square foot to erect, strip, carry, hoist, and patch ready to erect again. The total of these two, material and labor, may be taken as the square foot cost for the job under consideration.

Against them must be set the cost of the steel forms, taken in the same way. The headers and shores on the lumber list will be retained. The actual quantity may be more or less according to the permissible span of the metal pans. Purchase price and labor to receive and handle will have to be figured on the quantity of lumber required. This will give the material cost, unless the forms are rented on a square foot basis, in which case the rental will have to be included.

Labor erecting and stripping may be the contractor's own figure for the work, or it may be a sub-contract figure for the work. Added to the material cost it will give a cost per square foot that can be compared with the cost for wood forms.

It is interesting to note here that few contractors have any idea of how much of the labor erecting and stripping is given to the floor forms, and how much to the shoring. Before accepting a statement that the proportion is any given figure the contractor should make a check on one or two of his own jobs, as the proportion will often be reversed in two different organizations.

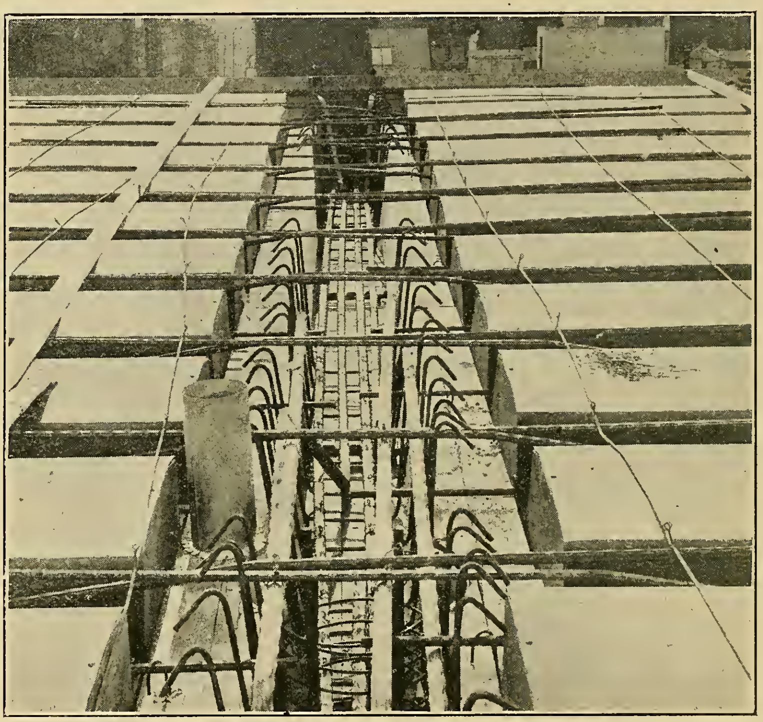

FIG. 1.

Items I (g) (h) (i). A job may require 200,00 ft. of lumber for floor forms, most of which will be used up afterward for curtain, parapet and penthouse walls, etc. 50,000 feet of new lumber may be needed in addition to complete the formwork. If metal forms are used it may be possible to avoid buying 100,000 ft. of the lumber that would ordinarily have been bought for floor forms, but it may happen that miscellaneous uses around the job would have ordinarily consumed 75,000 ft. of the 200,000 which would have been bought for wood floor forms, and which will have to be bought just the same for walls, stairs, etc. We would then have bought for metal forms the 100,000 ft. for shoring, plus the 75,000 for miscellaneous purposes plus the additional 50,000 ft. required in either case, making a total where steel forms are used of 225,000 ft. The cost of the extra lumber may more than offset the saving in labor and material for floor forms figured alone.

On one job that has been brought to the attention of the committee the standard 2 x 8 ft. metal floor panels were used successfully for the construction of a foundation wall. Ranger prices were braced temporarily in position, one on the ground, one four feet above the top of the footing and the top ranger eight feet above. The floor panels were then stood up against the rangers like boards in a fence, the usual spreaders being used to keep the forms apart, and tieing being done with wire around the rangers passed through the crack between adjoining pans. The pans were nailed to the rangers. The contractor's superintendent reported that using this method two men were able to set up forms complete for about 50 lin. ft. of wall per 8-hour day, or a superficial area of 400 sq. ft.

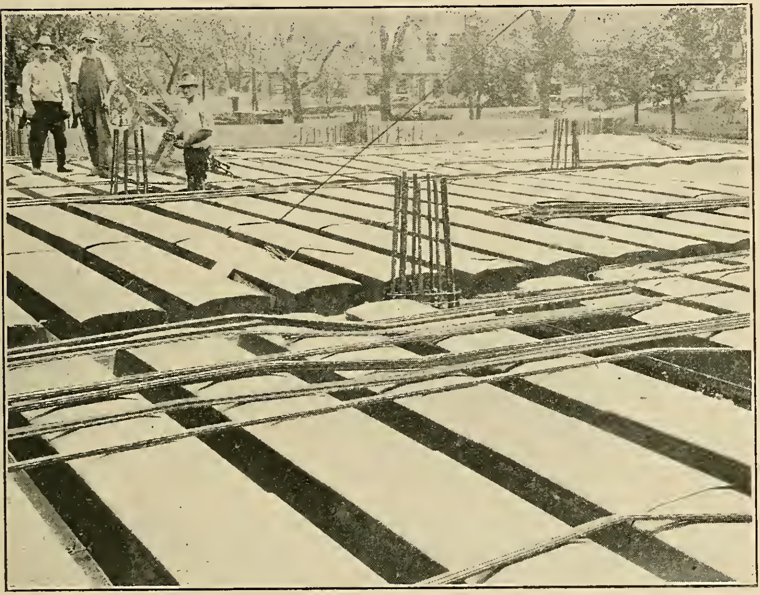

FIG. 2.

Items I (j) (k). Opinion among contractors seems to be unsettled on the question of superiority of ceiling finish obtained by the use of metal forms. The joints are less in number, and some feel therefore more conspicuous. There are no costs available to show whether there is any saving in pointing charges on ceilings. Efforts have been made with some degree of success to eliminate the joint between the pans by putting the crack with a plastic compound before the concrete is poured. The filler can be easily removed from the concrete after the pans have been lowered. If this is not done, and the joint-marks are felt to be objectionable, the easiest course would be to remove them with a rubbing stone or a wall scraper while the concrete is still green.

Any oil applied to the surface of the forms must dry up before the concrete is poured or it will be absorbed by the concrete. There has been some feeling that if an excess of oil were used the paint applied to the concrete afterward would be in danger of discoloration from the oil in the concrete. This point has been taken up both by correspondence with contractors and visits to buildings, and there is as yet no evidence to show that this has occurred. Metal column forms were in use for some time prior to use of metal floor forms, and there have been no complaints that the paint has been injured from this cause.

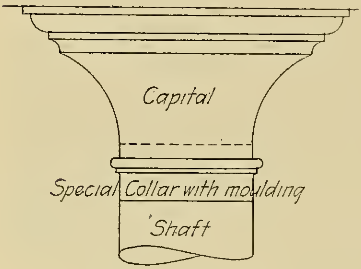

FIG. 3.

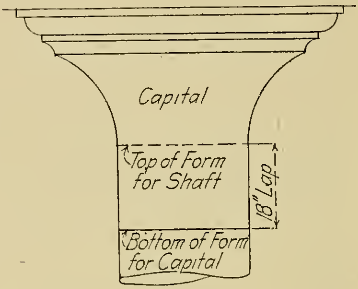

FIG. 4.

One thing in connection with pointing must be given attention, however. After a joint has been pointed the cement masons will brush off the patch lightly with a piece of canvas, or old cement bag, to make the edges of the patch less noticeable. When this is done a film of cement is deposited on the surface of the concrete around the patch. With the rougher surface given by wood forms this skin of cement will sometimes withstand the suction of the paint, but where metal forms are used, either for walls or columns, the paint will invariably pull this film away from the concrete. This will be found at times in wood form jobs as well and should be watched. Cement mortar applied in pointing seems to bond satisfactorily.

Metal-formed ceilings have been painted with and without primer, with equally good results. The use of primer as a precaution might be advisable where it is felt that an excess of oil was used to grease the forms. The paint should have the full proper quantity of binder.

The committee has been unable to find that anything has been done as yet in the way of putting a practicable beam-and-girder form system of metal on the market. The development of the so-called beam-and-joist system has been noted, and a brief description is appended for the benefit of those who are not aware of what has been done in this regard.

The beam-and-joist system follows the lines of the various types of permanent pan systems, in which the pans or domes are left in place to first support the green concrete and later to serve to some extent as reinforcement. A very light concrete construction was obtained with this construction for certain types of structure. The domes however were left in place, adding an element of extra cost, and being of very light metal were easily damaged while placing reinforcement and laying runs.

The pans are now made of much heavier metal to withstand rough handling and the incidental supporting formwork is so designed that the pans may be removed after the concrete has set. This system gives a beam or girder between columns in one direction, with the "joists" spanning the length of the bay between the beams in the other direction. The pans are made 6, 8, 10 and 12 in. deep and of standard lengths. The soffit of the joists is formed by plank or 6 in. structural channels laid flat on which the sides of the pans rest. The ends of the pans abutting on a beam are closed; the space between the end pans is filled with stretchers, similar to the pans but open on the ends, and the whole system is made tight by fitting "cover" pieces over the pans at the joints. Fig. 1 is a view looking along one of the beams and shows the pans in place ready for the concrete. The shoring required is quite simple, the system being capable of spanning up to 6 and 8 ft. without intermediate support. Complete details of the different methods of assembly may be obtained from any of the firms making these pans, whose systems of course vary in certain points.

CRITICISMS ANSWERED.

In last year's report note was made of certain criticisms of metal forms. These objections and the answers submitted by manufacturers are given herewith.

Sagging and Bulging of Forms.—This was due primarily to the use of a metal too light for the loads and to an insufficient number of ribs or stiffeners. This has been overcome by increasing the number of ribs and the use of a heavier metal.

The question of the effect of oil on the treatment of the finished concrete has already been discussed under Items I (j) (k).

Inserts may be held down by bolting, or by drilling two small holes between the ears of an insert and wiring down, or by driving barbed nails through. It is generally claimed that this part of the work has not given any trouble.

The joint between the column shaft and capital has been given considerable thought by the committee, and two drawings are given in which means are suggested for overcoming the objectionable necking-in so often found after the column has been stripped. This seems to be caused by the weight of the green concrete in the capital, which deflects the flared portion of the head just enough to distort the joint where the capital joins the shaft. It is suggested that this might be overcome by the use of a modified form of head, which would overlap the shaft about 18 in. or 2 ft., to provide the requisite stiffness. Another method would be to use a form of capital with a molded necking, which would lend additional lateral stiffness at this point and make any irregularity less conspicuous. This form of capital would probably be more expensive than the one with the extended neck. (See pages 3 and 4.)

In conclusion the committee wishes to emphasize the need for a careful study of conditions for each job, giving consideration to the factors that on that job have particular importance, before reaching a decision as to the kind of forms to be used. Conditions in the winter of 1921-1922 are not in any way comparable to the condiitons existing in the winter of 1919-1920 as far as material prices, labor supply or efficiency, etc., are concerned. As conditions change the contractor must be alert to re-study his methods and adapt his course as indicated by an impartial analysis of facts.

C. J. ABBOTT,
H. A. DAHLEN,
R. T. HARRIS,
C. E. NICHOLS,
J. P. TURNER,
W. F. LOCKHART, *Sec'y*,
E. A. STEELE, *Chairman*.

DISCUSSION.

J. G. Ahlers.—I want to speak as a general contractor of buildings,—industrial concrete buildings. The contractor's first problem is to get his formwork done as economically as possible, and in the severe competition we have now, we naturally look for every possible means of getting economy in construction work. We have studied carefully the sheet metal forms as applied to the flat slab as against the use of wooden forms. I have read carefully the report by the committee, and I believe by applying the outline given that most contractors at the present time will come to the conclusion that we have come, that there is no real economy in the use of flat-slab steel forms. **Mr. Ahlers.**

The reasons are two.

In the first place, I believe we deceive ourselves by comparing unit prices for flat-slab forms given by metal form contractors with the unit prices we all have as our cost records of the work. The panels themselves are the very cheapest part of the formwork on an industrial building. When you sub-let this part of the work I believe the contractor would find that the rest of the work will run up so high in cost that there is no saving.

In the second place, the contractor sub-letting the flat panels loses control of this part of the work and is not able to shift his crews and get full efficiency out of his organization. The sub-contractor is always looking for his own interests first in the matter of co-operating and coordinating, the general contractor comes second.

Wharton Clay.—J. J. Early who has made a deep study of the control of shrinkage in concrete and stucco is using expanded metal lath for the form work on the Parthenon building which is being reconstructed at Nashville, Tenn. The building is a very large reinforced concrete structure, reproducing identically the Parthenon of Athens. For his column forms he is using expanded metal lath extending about 8 in. between 2 x 6 in. timbers, 24 of which radiate around the columns. With this form he is able to pour concrete in a very workable mix and pour it cheaply on account of its plasticity, but secures ultimately the same results as if the concrete was poured in a very stiff mix with very little water. The excess water comes out through the meshes. **Mr. Clay.**

The same expanded lath is used in the lintels and other formwork. The lath is removed in this case and the rough surface stuccoed. It is found to be very much cheaper than the wooden forms which he used in the beginning of the job. This practice bears somewhat on the subject of metal mesh exterior reinforcement which is to be discussed in connection with fireproofing in the Joint Committee report tomorrow, and when surface reinforcement is desired for special fire protection when silicious aggregates are used, this method makes a radical reduction in the cost. It prevents spalling in case of fire in such aggregates.

Mr. Irwin. A. C. IRWIN.—Metal lath has been used for many years as a form for concrete. The openings in the mesh are not large—about ½-in. or less. There are at the present time several systems of constructing concrete houses utilizing metal lath as forms and many important structures have been built in this manner. Another example may be found in the columns of the replica of the Parthenon now under construction at Nashville, Tenn.

Those who have not witnessed work of this sort might naturally believe that the mortar would escape through the mesh. I have carefully observed the casting of floors and beams directly on metal lath as forms and there is little dripping of water noticeable. The particular thing, moreover, about the water is that what does drop through is practically clear and carries no cement with it. I would rather credit the use of metal lath as forms with requiring a plastic or jelly-like mix of concrete and thus cutting down the amount of water that can be practicably used, than to say that the metal lath itself by some mystic means extracts the water from the concrete that ought not to have been there in the first place.

Mr. Nichols. C. E. NICHOLS.—One reason, I believe, why I was appointed to the metal forms committee is that the organization with which I am employed had used them on several jobs. The jobs were under way at time of rising prices, and we adopted the metal forms because the prices we were able to get from the manufacturer compared favorably with our estimated cost for wood forms. During a period of rising prices, the tendency naturally is to protect your estimate, and we could see no way we could lose by using the metal forms, and possibly we might save money. We have used these now on four or five jobs and I think the costs came out as well as they might have with wood forms. However, the testimony of superintendents and of the construction men who followed the work is that under similar conditions they doubt if they would use metal forms again. I know somewhat the situation that the metal form manufacturers are in; they will talk quite glibly about economies, the ease with which metal is used, etc., and they will estimate the cost of wood forms to any contractor about as closely as he can; then they will set their prices just as high as they can possibly get them and sell the job. This testimony is not given with the idea of discouraging anyone from using metal forms; I think they are the coming thing in the future. But I think the one thing that must be done is for the metal form manufacturers to get their costs and their prices down.

I should like to mention one point which is covered in the report, that of fastening inserts to metal forms. "Inserts may be held down by bolting or by drilling two small holes between the ears of an insert and wiring down, or by driving nails through. It is generally claimed that this part of the work has not given any trouble." I may say here that that statement, "it is generally claimed," comes from the metal forms manufacturers. In personal discussion with contractors who have used metal forms and have been up against that problem, they are pretty universally of the opinion that a great deal of trouble has been given.

REPORT OF COMMITTEE P-1, ON STANDARD BUILDING UNITS.

(Including Block, Brick, and Structural Tile.)

Our committee consists of the following members:

Chairman, Robt. F. Havlik, Mooseheart, Ill.

Secretary, W. R. Harris, 542 Monadnock Block, Chicago.

W. H. Carey, Carey Concrete Company, Wisconsin Rapids, Wis.

Austin Crabbs, The Cement Products Co., 207 W. Fourth St., Davenport, Iowa.

H. G. Davis, Brooklyn Crozite Brick Corp., 140 Cedar St., Brooklyn, N. Y.

J. K. Harridge, Hydro-Stone Co., 15 E. Van Burean St., Chicago.

H. K. Holsman, 310 S. Wabash Ave., Chicago.

F. J. Kinsinger, Cast Stone Block & Machine Co., Ltd., Windsor, Ont., Can.

C. E. Lindsley, 882 Clinton Ave., Irvington, N. J.

J. W. Lowell, Universal Portland Cement Co., 210 So. La Salle St., Chicago.

C. A. Steward, Plano Cement Products Co., Plano, Ill.

S. Walker, Lewis Institute, Madison and Robey Sts., Chicago.

S. H. Wightman, 305 Monadnock Block, Chicago.

We have held seven meetings in the office of the Secretary, on the following dates: Sept. 23, Oct. 6, Oct. 14, Nov. 4, Nov. 22, Dec. 2, and Dec. 16. Careful minutes have been kept of each meeting and copies of these were mailed promptly after each meeting to both sitting and absent members of this committee. Copies of all communications bearing on the work of this committee which were received by the chairman and secretary of the committee were sent promptly by the secretary to the committee members.

At the second meeting held Oct. 6, it was decided that preliminary drafts of the proposed specifications be prepared by the following: Mr. Harris on concrete brick, Mr. Wightman on concrete building tile and the chairman on concrete blocks. These were prepared and submitted to the committee at the following meeting on Oct. 14. At the next meeting, it was found that many sections were repeated in each of the three specifications so it was unanimously decided by the sitting members to combine the three sepcifications into one. The revised combined specifications for the above three products were re-submitted to the committee at the meeting held Nov. 22. After careful consideration it was decided to send these specifications as revised to all members of the committee for letter ballot. This was done. At the Dec. 16th meeting, minor changes in phraseology were made and the ballots of the absent members and the votes of the sitting members were canvassed with the result that nine members: Messrs. Crabbs, Harridge, Harris, Havlik, Holsman, Kinsinger, Lindsley, Steward and Walker voted for the combined specifications; and three members, Messrs. Lowell, Carey and Wightman, voted against the specifications and one member, Mr. Davis, did not vote.

A copy of these specifications was sent to the secretary of the Institute, who sent out preprints thereof to all members thirty days prior to the Annual Convention in accordance with our By-Laws.

These specifications are submitted herewith* for consideration of the Institute so that they may be acted upon by letter ballot by our entire membership.

A lively discussion arose within the committee as to whether the strength of concrete building units should be considered on the basis of gross or net area of the units. An overwhelming majority of the committee favored figuring the strength on the gross area basis as has been done since the Institute was organized and the first specifications were adopted. No special provision is made for cinder concrete units because adequate information concerning these is lacking but it is hoped that during the coming year enough information on such units may become available so that special provision can be made for these should it seem wise to do so.

ROBT. F. HAVLIK, *Chairman.*
W. R. HARRIS, *Secretary.*

MINORITY REPORT OF COMMITTEE P-1

January 16, 1922.

Secretary, American Concrete Institute:

The following is the Minority Report of Committee P-1, American Concrete Institute, on Standard Building Unit Specifications:

The minority made an earnest effort to have consideration given their views during the discussions of the committee. The last meeting of the committee developed a serious difference in point of view of such a radical character that the majority report is wholly discordant with the views entertained by the minority. If this were merely a question of difference in viewpoint as to formal matters, it would be of only passing interest. The facts are, however, that the differences are of such radical character, and the consequence of the adoption of the majority report would be so highly injurious to a most important part of the concrete building unit field, that the differences are deserving of most careful consideration and investigation on the part of the members of the Institute.

In fact, points of view were so widely variant that even a reformation of the majority report will not, in the opinion of the minority, cure the defects in the majority report. Consequently the first recommendation which the minority make is that, in addition to the amendment to the majority report hereinafter set forth, the work of the committee shall be continued for another year; the membership of the committee enlarged and careful attention paid to the appointment on the committee of men of widespread engineering experience, who are capable of grasping the requirements for different types of structural units, and whose judgment will be backed by actual experience in the field.

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*Printed on p. 312 of this volume.

No more useful work can be attempted by the American Concrete Institute than the formulation of standards and specifications which shall serve as a guide to the industry as a whole. On the other hand, if the work be so ill advisedly done, and be not maturely considered, then great, if not irreparable damage, may be caused. This is the nub of the minority contention with respect to the action of the majority of the committee.

It is not a question of mere overriding of personal views, but it is a question of such an unreconcilable difference in conception as, in fact, amounts to an absolute strangulation of the light-weight concrete structural industry, should the views of the majority prevail, which industry in the past several years has assumed very great importance and bids fair to place the concrete structural units in the field of both light and heavy building construction, as has not heretofore been the case with respect to other types of concrete structural units.

The minority objection to the majority recommendations is of a two-fold character, first: formal; second: substantial. Taking up the formal objections first:

The minority consider the specifications, as drafted, to be incomplete, wrong in principle, in a form not acceptable to the various building departments, and injurious to the concrete products industry, if permitted to be spread broadcast with the endorsement of the American Concrete Institute.

The minority is of the opinion that the formal vices which are found in the form of specifications submitted by the majority, cannot be entirely cured by amendment on the floor, and that, to put the specifications in proper shape, the work of the committee must be continued for another year.

Second: As to substance, the minority contend that the specifications as drafted, rankly discriminate against light-weight structural units, which are commonly designated concrete building tile. If the specifications as recommended by the majority are to be adopted and are to be strictly enforced, concrete building tile would be cut out of the two most prominent fields of construction, which are small residence and small building construction, such as garages, one and two-story office and commercial buildings, etc., and on the other hand the heavier load construction, such as four and five-story apartment houses, factory buildings, etc.

At the outset the minority wishes to make clear a point which unquestionably has been confusing in the minds of some members, that is, that we are not in favor of reducing the quality of concrete permitted to be used in structural units. On the contrary, we are in favor of increasing the requirements as to quality, so as to necessitate the use of a quality of concrete which cannot but prove satisfactory. All that we ask is, that the test be applied scientifically with respect to the practical characteristics of the units to which same is to be applied. We contend that the light-weight concrete structural unit cannot be classed, with justice to such unit, with the heavy wall concrete block, and that insistence upon such classification practically eliminates building tile, or light-weight units, from other fields of greater usefulness.

The majority report places all units, regardless of characteristics, on the same basis, namely: a flat test of 1000 lb. per sq. in., with permissible testing as low as 700 lb., provided the average be 1000 lb.

With respect to quality of concrete, we recommend that all structural units, other than brick, shall average 1500 lb. per sq. in. of net section with no one test falling 10 per cent under same. We know of no other structural material that is allowed a variance of 30 per cent, as proposed in the majority test. We regard that as unnecessarily lenient and as setting a standard of permissible variance which should not be set.

We believe 10 per cent permissible variance to be fair, and to place concrete units on the same plane of uniformity of quality which other competitive materials are compelled to meet.

We contend that the application of the 1000 lb. gross area (one standard) specifications to all classes of concrete products, automatically eliminates light-weight building tile units, which hold the greatest promise for future development in the entire field of concrete structural units. Take, for example, small building constructions, such as residences, etc., which is a field in which the heavier type of concrete unit as represented by the concrete block, has in the past years obtained less than 1/10 of 1 per cent of this class of construction. This, as every one who has had any familiarity with the promotion of concrete units, knows, has been due to the heavy weight, excessive cost and inferior quality of concrete used.

Effort to promote the use of concrete block for house construction, taking results as a whole, and conditions throughout the country as a whole, has been a dismal failure.

We believe, as in fact do most men in touch with concrete structural unit development, that concrete units properly made, for respective intended use, can serve the entire field of masonry construction, whether same be in the class of heavy duty masonry construction—wherever clay brick masonry is used—which may be six stories in height down to the smallest light wall construction, such as one or two-story residences, commercial buildings, garages, etc.

The class of concrete structural units, known as concrete building tile, is a type of unit designed to be used as a brick replacement unit, and which, because of the character of its design, can be varied as to compression strength, so as to cater to either the heavy loading or light duty fields.

The heavy duty wall may, perhaps, be loaded from 150 to 200 lb. per sq. in., while smaller buildings seldom require over 20 to 30 lb. per sq. in.

It is, in the opinion of the minority, ridiculous to assume that only one type of concrete product shall be made to serve either the one field or the other. To require a heavy duty building tile to be made, where it is intended to be used only in light duty walls, is to impose a tremendous handicap upon building tile, and would practically eliminate same from consideration in competition with clay products and lumber.

On the other hand, to say that the strength requirement shall be 1000 lb. per sq. in. gross area, as such provision is usually interpreted, means that with the tile unit confined to a safety factor of either 8 or 10, would limit it to either 100 or 125 lb. per sq. in. permissible loading. It is obvious that this would cut tile out of the very desirable heavy duty loading, which, as stated, may run from 150 to 200 lb. per sq. in., and for which the tile unit can be scientifically strengthened.

While these specifications are not intended as building code standard, the minority wishes to point out that it only remains for a factor of safety to be added to the specifications to create what is a building code regulation as regards allowable loading.

Furthermore, it has been stated that the specifications will be acceptable to building departments—certainly they form the basis upon which building code regulations will be made. This is precisely the vice against which the minority protests, in that any such enforced grouping will, with the endorsement of the American Concrete Institute, be looked upon with favor by various building departments, and irreparable injury will be done to the entire concrete building tile development. Injury is an injury, whether it proceeds from good or bad intentions, and whether it proceeds from the hand of friend or enemy. For concrete building tile to be stifled and the field of usefulness be largely decreased, if not entirely destroyed at the hands of those who should be friendly to such units, namely, the American Concrete Institute, is both ridiculous and tragic; tragic because of the consequences, and ridiculous at the idea of those in the industry not having foresight enough to recognize the damage they are doing to a promising field of concrete products development. Certainly, we cannot expect the clay or lumber interests to be more charitable towards concrete structural units than the concrete industry itself proves itself to be.

We illustrate a few of the outstanding flaws in the 1000 lb. gross area specifications, and what same will mean if adopted as the prevailing view of the American Concrete Institute.

Suppose, we assume, a factor of safety of 8 will be applied in building regulations as regards the 1000 lb. gross area section, this would mean that concrete products could be loaded up to 125 lb. per sq. in. and no more. No more for the reason that if the concrete industry does not regard concrete units as capable of meeting a more severe test, its enemies are certainly not going to permit any higher rating to be given.

In this connection those who are in touch with building code regulations know that clay interests are behind the revision of building codes in a number of the largest cities at the present time, and are moving to have revision in other cities. They know what they want, and yet the one unit which can successfully compete with clay units in the field of lighter construction is being legislated out of existence by its so-called friends. Surely this is an anomalous position.

Concrete products, whether they be solid or hollow (cored) construction, averaging 1500 lb. per sq. in., should be permitted to carry loads 50 per cent in excess of the 125 lb. limit, but the 1000 lb. gross area specifications would close the door to this in any construction which required loading in excess of 125 lb. per sq. in. It is obvious that if the factor of safety should be placed at 10 then 100 lb. loading would be the limit, while the usual allowable loading on clay brick is 200 lb. per sq. in.

On this point it is the contention of the minority that the specifications should recognize specifically that concrete units exist and can be developed to produce a structural unit at a competitive cost with other materials, which can stand up to 200 lb. per sq. in. loading.

On the other hand it may be desirable to build a small building of a unit which is 8 or 12 in. in wall width, having say 60 per cent net load carrying section, the same testing 1500 or 1600 lb. per sq. in., such unit could not meet the required 1000 lb. gross area test, and would be rejected, even though it undoubtedly would have a factor of safety as regards loading of at least 40 and possibly even 100.

There have been manufactured for several years structural units which have only from 20 per cent to 33 1/3 per cent net section. Four such types of well known commercial structural units, all coming within the class of what is known as building tile, were offered by the minority as evidence of its position at one of the committee meetings, in order to show the merits of same, and likewise how impossible it would be to commercially make any of these units stand the 1000 lb. gross area test.

In this connection it must be understood that no unit is entitled to a higher rate than the section which it can show in actual compression. The question is altogether different from one where the entire surface is theoretically figured. No modern building department which is worthy of its name is going to permit a rating based on a percentage of section which does not actually carry loading.

One type of the four examples placed before the committee has been manufactured in Detroit for years, and some forty million have been used. Over two and one-half million of this particular form of tile were used in school buildings during 1921. This tile was demonstrated to the committee to have only 25 per cent net bearing section. The same tile have been manufactured for over seven years, and have always averaged better than 1500 lb. per sq. in. The Detroit building code and the building department have permitted their use wherever brick were used, and have permitted the loading up to 200 lb. per sq. in. net section, but taking this same tile which has been used with such complete satisfaction for so many years on the 1000 lb. per sq. in. basis, and realizing that there is only 25 per cent net section, this same tile would have to stand 4000 lb. per sq. in. Recognizing the fact that a structural unit will not test up to its full concrete strength would mean that the concrete of which the tile was manufactured would have to be made to stand at least 5000 lb. per sq. in., consequently the 1000 lb. gross area test would absolutely eliminate this tile in its light-weight shape, and would compel the makers thereof to at least double the web thickness of the tile.

Of the three other types shown to the committee, each one would likewise have to have more than double its web thickness, or else its manufacture would have to stop.

While the 1000 lb. gross area method would permit a loading of 500 lb. per sq. in. net section on the above tile providing they met the 1000 lb. gross area test, probably more than would be conservative, the type of construction in which they are intended to go, would probably require less than 30 lb. per sq. in.

It is the contention of the minority that while the gross area method of computation would permit a loading of up to 125 lb. per sq. in. gross area, the type of construction for which the light-weight tile is ideally adapted, does not, as far as wall duty is concerned, require over 30 lb. per sq. in. gross area, consequently the one specification method mentioned, namely 1000 lb., is just as disadvantageous on the light end as it is on the heavy end, because it requires the development of a strength greatly in excess of that which proposed usage requires, and places the tile under an enforced manufacturing disadvantage, since it naturally runs up the cost of making same without any resultant advantage.

While other examples similar to the above can be given almost without end, it is a fact, as the minority see it, that the one standard method of specifying strength requirements in concrete structural units is basically wrong. It lowers the allowable loading to 125 lb. as regards heavy construction, and it kills the light-weight unit entirely. In spite of the fact, there are a half dozen different types of unit on the market, all of which are capable of entering this field, and are measuring up to the requirements for light-weight construction, and this, in spite of the fact that on the light-weight side it represents the field of biggest promise, and although as shown in the Detroit example, which can be duplicated in other cities, millions of these lighter weight units, that could not meet the 1000 lb. test, have been used in types of buildings not requiring the 1000 gross area standard, and used without a single failure and with entire satisfaction for many years.

The minority further contends that it is possible in requiring a 1500 lb. minimum strength, or net section test, to insure a good grade of concrete. It is a better guarantee of a higher quality concrete than is the case with the 1000 lb. gross area allowable by the majority report, which even may fall as low as 700 lb.

It is the opinion of the minority that the gross area test is unscientific, is not in accord with sound practice and is entirely at variance with the method of computing stresses as regards other classes of building material, such as steel, reinforced concrete, etc. The preferred method of specifying the strength requirement for structural units is a net section basis, with a factor of safety of 8 or 10. This for a 1500 lb. quality of concrete, would permit a loading up to 187 lb. per sq. in. for each square inch of net section.

How this would work can be simply demonstrated. In the case of solid block, etc., the loading on gross area would be a permissible loading of 187 lb. per sq. in. Where, however, the load carrying section represents only one-third of the gross area the allowable loading would be only one-third of 187 lb. or 63 lb. per sq. in. gross area and so on.

It is the claim of the minority that the adoption of a net section basis of computation is the fairest of all methods, and that it automatically and scientifically takes care of the various gradations of duty on the one hand and permissible loading on the other. All that is necessary to do on the part of the manufacturer of concrete structural units is to increase or decrease the available net section to correspond with the loading requirements. For this tile is ideally adopted. Since it can be done and done consistent with sound engineering and with perfect construction safety, why should it not be permitted to be done? Why should the industry impose a barrier to its being done,

However, the minority is not insistent upon its point of view regarding the net section, if general opinion does not sustain same. It does contend, however, that if the gross area method of testing is to be retained, then there must not be less than four classes of tests in order to cover a wide range of products in the various construction fields. These may be designated as follows:

Class	Compressive strength per sq. in. Gross area	Allowable Loading
Heavy Bearing	1200	150
Medium Bearing	800	100
Light Bearing	480	60
Non Bearing	250	Sufficient to permit handling

The minority recognizes that it is not the function of Committee P-1 to draft building code specifications; none-the-less, it is impossible not to recognize the bearing of these specifications upon building code practice, and that the specifications must provide the technical foundation for the code.

The minority further unqualifiedly state that there has not been that full discussion of conditions of that class of construction for which concrete building tile in particular are adaptable, as should be given in formulating standards which must determine the future of such units.

While the minority feel that the work of the committee, as represented by the majority report, is of little, if any, improvement over the report of the committee for the previous year, and in fact makes few changes of moment in such former specifications, it does not believe it necessary to go to the extreme length of asking for the rejection of the entire report, provided the same can be amended, in accordance with the following specific recommendations.

The matter of greatest importance in the opinion of the minority is that the one standard method of classifying products regardless of the use to which they are to be put, will work an unmeasurable harm to that class of units designed to carry loads in excess of 100 lb. per sq. in., and to light-weight units designed for residential work and like construction. The minority is not so much concerned as to just what form the specifications shall take as long as they provide a safe and fair basis whereby the merits of all classes of products are recognized and classified.

The minority therefore are not offering any specific amendments but prefer to have the members earnestly consider the entire matter, and after careful consideration, to make such amendments as will recognize or properly govern all classes of products. The minority recommend, however, if the gross area method of classification is used, that no less than three classifications for bearing walls will suffice.

S. H. WIGHTMAN,
C. A. STEWARD,
W. H. CAREY,
H. A. DAVIS,
AUSTIN CRABBS.

[Following the presentation of the majority and minority reports, there was a long discussion involving the merits of the two views submitted, at the end of which the meeting voted that both reports be printed in the Proceedings but that both be referred back to the committee for further consideration. In the course of the discussion John W. Lowell presented different suggestions for proposed standards which had been developed in an effort to reach a compromise. Sections 3 and 4 of the proposed alternatives by Mr. Lowell are printed following the proposed specifications on the next page. These are printed here because they offer a new fundamental basis for the standards presented in both majority and minority reports.—EDITOR.]

PROPOSED STANDARD SPECIFICATIONS FOR CONCRETE BLOCK, BRICK AND BUILDING TILE.*

1. The purpose of these specifications is to define the requirements of concrete block, brick and building tile for use in building construction.

2. *Concrete.* The word concrete as used in these specifications shall be considered as meaning portland cement concrete.

3. (*a*) *Concrete Block and Building Tile.* Concrete block and building tile for hollow and solid walls must be subjected to (a) compression and (b) absorption tests.

(*b*) *Concrete brick.* Concrete common brick must be subjected to (a) compression tests. Concrete face brick or concrete common brick used as face brick must be subjected to (a) compression and (b) absorption tests.

4. *Ultimate Compressive Strength.* (*a*) Hollow and solid concrete block, two piece building block brick and building tile for load bearing and exterior walls.

The ultimate compressive strength of such units when delivered to building site or at twenty-eight (28) days must average not less than one thousand (1000) pounds per square inch of gross cross-sectional area of the unit, as used in the walls, and must not fall below seven hundred (700) pounds per square inch in any test.

(*b*) Solid or hollow concrete block, two piece block and building tile for non-load bearing and partition walls.

The ultimate compressive strength of such units when delivered to building site or at twenty-eight (28) days must average not less than three hundred (300) pounds per square inch of gross cross-sectional area of the unit, as used in the wall, and must not fall below two hundred (200) pounds per square inch in any test.

5. *Gross Cross-Sectional Area.* (*a*) Concrete block, brick and building tile. The gross cross-sectional area shall be considered as the product of the length by the width of the unit. No allowance shall be made for the air space of hollow units.

(*b*) Two-piece concrete block. The ultimate compressive strength in pounds per square inch of gross cross-sectional area of each unit as used in the wall shall be regarded as the ultimate compressive strength of the unit divided by the product of the length in inches times one-half the thickness in inches of the wall for which the block is intended if the units are equal in area. When the units are not equal in area, the combined ultimate strength of each pair of units shall be divided by the product of the length in inches of one block times the full thickness of the wall for which the blocks are intended.

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*Submitted by Committee, P-1, on Standard Concrete Building Units, to supersede a part of Standard No. 10, published in *Journal*, A. C. I., Vol. I, January, 1915.

6. *Compression Tests.* Compression tests shall be made as follows:

(*a*) Samples to be tested must be dried at a temperature of not less than seventy (70) degrees F. for three (3) days or to a constant weight at a temperature not to exceed two hundred and twelve (212) degrees F.

(*b*) The samples must be carefully measured and then bedded in plaster of paris or other cementitious material in order to obtain uniform bearing and loaded to failure.

(*c*) Load shall be applied through a spherical bearing block or other means whereby test loads will be uniformly distributed over the sample being tested. When testing other than rectangular units great care must be taken to apply the load at the center of gravity of the specimen.

7. *Absorption.* (*a*) Concrete block and building tile for load bearing and exterior walls.

The absorption of such units when delivered to building site or at twenty-eight (28) days must not exceed ten (10) per cent when tested as hereinafter specified.

(*b*) Face brick or common brick used as face brick.

The absorption of such units when delivered to building site or at twenty-eight (28) days must not exceed ten (10%) per cent when tested as hereinafter specified.

8. *Absorption Tests.* The sample shall first be thoroughly dried to a constant weight at a temperature not to exceed two hundred and twelve (212) degrees Fahrenheit and the weight recorded. Then the sample shall be immersed in clean water for a period of forty-eight (48) hours. The sample shall be removed, the surface water wiped off and the sample reweighed. The percentage of absorption shall be regarded as the weight of the water absorbed, divided by the weight of the dry sample and the quotient multiplied by one hundred (100).

9. *Samples.* At least three (3) samples must be provided for the purpose of testing. Such samples must represent the ordinary commercial product. Whenever possible, the tests shall be made on full sized samples. In cases where the material is made and used in special shapes and forms too large for testing in the ordinary machine, smaller specimens may be used.

SECTIONS 3 AND 4 (SUBMITTED BY JOHN W. LOWELL).

3. Concrete block, brick and building tile must be subjected to compression tests. Where such products are to be exposed to the weather, they must be subjected to (a) compression tests and (b) absorption tests.

4. *Ultimate Compressive Strength.*

Concrete block, brick and building tile.

The ultimate compressive strength of such units at the time when laid in the wall or at twenty-eight (28) days must comply with the requirements set forth in the following table:

Unit	Compressive Strength Requirement of Unit in Pounds Per Square Inch of Gross Area		Allowable Load Pounds Per Square Inch
	Ave of all tests	Indiviual min.	
Brick	1500	1200	250
Heavy Bearing Block or Tile	1200	1000	200
Medium Bearing Block or Tile	700	600	75
Light Bearing Block or Tile	500	400	40
Non-Bearing Block or Tile	250	....	Sufficient to handle

In no case shall concrete for block or tile for bearing wall construction have a compressive strength of less than 1000 lb. per sq. in., based on the net area of the block or tile as used in the wall.

DISCUSSION. REPORT OF COMMITTEE S-1, ON REINFORCED-CONCRETE CHIMNEYS.*

By Edward Godfrey.

Referring to the tentative draft of specifications for reinforced-concrete chimneys, I am in agreement with most of this but take exception to some parts and think it is incomplete in some essential respects.

Paragraph (4), under Design, is severe. Upright rods could well be spaced more than 6 in. apart in many cases and 2 in. from the surface would be sufficient in many cases for the placing of rods. I suggest 12 in. for the first figure.

Paragraph (5) is indefinite. The pressure of a 100-mile wind doesn't mean anything definite. I suggest that a flat wind pressure of 40 lb. per sq. ft. on surfaces normal to the wind and 20 lb. per sq. ft. on the diametral projection of a cylindrical surface be specified.

The specifications are too general and indefinite in the matter of design. It would be better to make no report than to leave so many things at the option of the designer, especially in a field where matters of design are so loose-jointed as this one.

I suggest the following additions which are among recommendations that I submitted to the Secretary of the Concrete Institute at the Convenion in 1920.

To find the pressure per square foot due to the wind moment on the base of the chimney—Let

M = the wind moment in ft.-lb.
U = soil pressure per sq. ft.
S = side of square base in ft.
D = diameter of round base in ft.

Then

$$U = \frac{6M}{S^3} \text{ or } \frac{32M}{\pi D^3}$$

U is to be added to the dead load per sq. ft. on the soil.

If U should be found to equal or exceed the pressure per square foot due to the dead weight of the chimney, the stability of the chimney is endangered, and more weight must be added in the base.

If it is necessary to use a wide spread in the base or foundation, projecting more than three-quarters of the thickness of the slab beyond the chimney, reinforcement will be needed in the slab. This reinforcement should consist of rods lying near the bottom of the slab in both rectangular directions and in both diagonal directions, all rods lying under the cylinder of the chimney. For a conventional way of finding the stress in these rods use the following. Find the total area of the base less the area of the chimney cylinder. Multiply one-eighth of this by U, the unit pressure allowed on the soil. Multiply this by one-half of the distance which the slab projects beyond the chimney cylinder plus six inches. This is the cantilever moment on one set of projecting rods. Divide this by seven-eights or the depth of the slab from its top to the plane of the steel to find the stress on a set of the rods.

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*See p. 221 of this volume.

The anchorage of the chimney into the foundation will be effected by extending the vertical rods into the base for a distance of at least 50 diameters of the rods. The size and spacing of these rods will be found as for the cylinder of the chimney, by the method described later.

To find the unit compression on the concrete of the chimney shell, first estimate the weight of the shell, itself, above any given horizontal section and divide this weight by the area of the section in square inches. Add to this unit the extreme fiber stress due to bending from the wind moment, calculated in the following manner:—Let

$12M$ = wind moment in inch-pounds at the section considered (M being in foot-pounds)
d = outer diameter of shell in inches
d' = inner diameter of shell in inches
u = unit compression at extreme fiber on leeward side.

Then

$$u = \frac{122.2Md}{d^4 - d'^4}$$

The extreme fiber stress, u, added to the unit compression due to the direct load gives the total compression unit on the concrete.

When there are openings in the side of the chimney, the shell should be thickened up beside the opening, by pilasters making up the area of the cross section lost in the opening. Also vertical rods should be used beside the opening at least equal in area to those that would be used in that portion of the cylinder. These rods should be straight and lapping the rods that occur over and under the opening by 50 diameters.

Above the opening there should be a lintel reinforced with straight horizontal rods, and capable of distributing the vertical shell load into the pilasters beside the opening.

If the opening is not at the base of the chimney, there should be a similar lintel below the opening, reversed in position to the one above it and reinforced in a similar way.

For vertical tension reinforcement, it is recommended that rods be used capable of taking the tensile stress that a sector of the concrete would have to take in a plain concrete chimney; that is, that the neutral axis of the cylindrical shell be taken at the center of the circle. The tension on a rod is then found in the following manner:—Let

$12M$ = bending moment in inch-pounds at section considered
n = number of rods in circle
W = weight of chimney above section considered
d'' = diameter in inches of circle of rods
s = tensile stress on a rod.

Then

$$s = \frac{48M}{d''} - \frac{W}{n}$$

It is recommended that for circular hooping an area of steel equal to about two-tenths of one per cent of the vertical sectional area of the shell be used.

Commenting again on the proposed specifications, I do not think that three-quarters of a minute is long enough time for batch mixing. It should be 1¼ or 1½ minutes.

REPORT OF COMMITTEE J-2, ON CONCRETE PIPE

(JOINT COMMITTEE)

Many of the railroads of the United States have for a number of years been using reinforced-concrete culvert pipe, a few manufacturing their own, others purchasing from outside pipe companies. Highway culvert pipe have also come into general use. Practically all parties interested whether they be manufacturers, designers or users have for sometime realized the necessity of coming to some definite conclusions as to designs, tests, etc., so that specifications would be standardized. The designer and the user wished to know what strength of pipe could be counted upon and the manufacturers what strength would be required. A thorough understanding of the problems involved and standardization of the governing features means elimination of unnecessary expense, and at the same time, assuring the required strength. In 1919 the Joint Committee on Reinforced-Concrete Culvert Pipe was formed, the members consisting of two from each of the following societies:

American Concrete Institute.
American Assn. of State Highway Officials.
American Railway Engr'ng Assn.
American Society for Testing Materials.
American Society for Civil Engineers.
American Concrete Pipe Assn.
U. S. Dept. of Public Roads.

The first meeting of this Joint Committee was held at Atlantic City in June, 1919, at which time the necessary organization was perfected and a program laid out. It was decided to divide the work into four sub-committees as follows:

No. 1—Classification and Loading Requirements.
No. 2—Design.
No. 3—Methods and Requirements of Tests.
No. 4—Workmanship and Installation.

Since that time several meetings of the full committee as well as the sub-committees have been held and much work accomplished.

A careful study of the subject very soon brought the entire committee to a realization that before much if anything could be done toward standardization of design, etc., we must know what to expect in the way of loads that might come upon the pipe. The Engineering Experimental Station of the Iowa State College at Ames, Iowa, had already studied the question of loads on pipe in ditches, and the results of their study are now incorporated in the American Society for Testing Materials standards. Arrangements were made for them to continue their work in order to determine, if possible, what to expect in the way of loads in highway and railroad embankments.

The tests so far made at Ames may be divided into two general classes. Those from August, 1919, to July, 1920, were based on determining loads from embankment as ordinarily built; in other words, the pipe was first laid in position on the ground and then dirt deposited in the form of an embankment. The experiments of July, 1920, to July, 1921, consisted of first building the embankment to partial height, then excavating the ditch in which the test pipe are placed, the ditch refilled and then the balance of the embankment placed as before. In each case the culvert was divided into sections 2-ft. in length, independently supported so that the maximum loads transmitted to each 2-ft. section could be measured. The total length of the culvert was 20-ft. and the external diameter 3½-ft. In addition, a number of Goldbeck pressure cells for determining unit earth pressure were placed in position above the top of the culvert to check the other measurements of vertical pressure. Beginning in August, 1919, an embankment 20-ft. in height was built up gradually and completed in October, 1919. The material used was a sandy loam top soil weighing 96-lb. per cu. ft. Pressures transmitted to the culvert from the embankment material was observed as the filling progressed, and after the completion of the fill two daily observations of pressures on each of the ten sections were made throughout the winter and until June, 1920, at which time the embankment was removed. When the embankment was taken down to the level of 12-ft. above the diameter of the culvert a truck loaded to a total of 10 tons weight was placed in various positions on the surface of the fill and the load transmitted to each 2-ft. pipe sections were noted. This practice was repeated at levels of 11-ft., 10-ft., 9-ft., etc., being down to a minimum of 3-ft. above the top of the culvert.

For the experiments of July, 1920, to 1921, an embankment was first built of sandy loam top soil to a level of 4-ft. above the top of the culvert. The soil was deposited in layers and a road roller was used to consolidate the embankment. After completion to this height of 4-ft. above the top of the culvert, a vertical trench 4½-ft. wide was dug immediately over the top of the culvert and extending down to the culvert itself. The trench was then refilled with material deposited in the loosest possible condition. The entire embankment was then built up to a height of 20-ft. above the top of the culvert, the work being completed in September, 1921. During this process it was found that the weights imposed upon the culvert by the embankment at different times were much less than those in the work of the preceding year. Up to a height of 12-ft. the loads obtained corresponded closely to those found on pipes in ditches of solid material in accordance with preceding investigations on the theory of loads on pipes in ditches. This special method of constructing culverts in embankments reduced the imposed loads to about two-thirds of those observed after using the ordinary method of making the fill.

As a result of these tests a considerable amount of data has been collected but the Joint Committee feels that additional work should be done along this line before making definite recommendations. The Experimental Station at Ames is experimenting further and is now making a study of the results so far secured and probably will prepare a bulletin during the summer which will be in the form of a progress report with tentative suggestions.

In August, 1921, Prof. A. Marston, Dean of the College of Engineering, Iowa State College, Ames, Iowa, was elected chairman of the Joint Committee to fill the vacancy caused by the resignation of Mr. Shelley as chairman. The committee feels that they were particularly fortunate in securing Prof. Marston's acceptance of this office in view of the very great amount of work and study that he has done on this question of loads on pipe.

The last meeting of the full committee was held in Chicago on Nov. 28, 1921, and as evidence of the interest that is being taken by the members we are pleased to report that each society represented on this committee had one or more of its representatives present. At this meeting the following reports of sub-committees were adopted as tentative:

Sub-committee No. 1—Classification and Loading Requirements.

1. It is recommended that a class be established in which the ultimate strength of the pipe by the sand bearing test in pounds per foot of length of pipe shall be 2250 x *D* where *D* is the nominal internal diameter of the pipe. (This class was not designated but was referred to as class "2250 *D*.") This is the pipe designed for ordinary highway purposes under fills up to 12 ft. high.

2. It is recommended that a class be established in which the ultimate strength of the pipe by the sand bearing test in pounds per foot of length of pipe shall be 3000 x *D* where *D* is the nominal internal diameter of the pipe in feet. (This class was not designated but was referred to as Class "3000 *D*.") This pipe was intended for ordinary minimum requirements of railway practice.

Sub-committee No. 2—Design.

1. It is recommended that the classification loads referred to by the Classification Committee shall be used as a tentative basis for the design of pipe.

2. It is recommended that no visible cracks shall appear under the test loads at a loading two-thirds that of the ultimate for pipe 24 in. in diameter or at 50% of the ultimate for pipe 84 in. in diameter; intermediate diameters to carry proportional percentages of the test load.

3. It is recommended that the minimum covering for the reinforcing steel be ¾ in. from the face of the steel to the nearest face of the concrete.

Sub-committee No. 3—Methods and Requirements of Tests.

Specifications will be prepared covering portland cement, fine and coarse aggregates, reinforcement and other materials entering the construction of reinforced concrete pipe. Whenever possible the committee shall adopt specifications of materials that have been standardized.

It is recommended that the committee adopt the relation between three point loading and sand loading tests of the American Society for Testing Materials.

Sub-Committee No. 4—Workmanship and Installation.

1. It is recommended that specifications prepared by this Sub-Committee shall be in the nature of an addenda providing advice to engineers.
2. It is recommended that the portion dealing with the manufacture be confined to finished product with the following provisional additions:

   (a) Hydration.
   (b) Age before transporting.
   (c) Methods of loading for shipment.

3. It is recommended that the portion dealing with laying methods outline recommended practice and co-ordinate such methods with the strength requirements provided by the other sub-committees.

Your members of the Joint Committee recommend that this Institute continue to be represented on this committee.

B. S. PEASE, *Chairman.*
A. B. COHEN

DISCUSSION OF REPORT OF JOINT COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

One day of the 1922 annual meeting was devoted to a discussion of the Tentative Specifications for Concrete and Reinforced Concrete which were submitted June 1, 1921, as a progress report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, on which the American Concrete Institute is represented. The method of conduct of the discussion was announced as follows by President H. C. Turner.

Mr. Turner. H. C. TURNER.—In referring this progress report to the Institute the Joint Committee adopted a rule as follows: "The initial report of the Joint Committee shall be considered by each of the five organizations as a tentative report submitted for criticism and discussion limited to not less than six months nor more than one year. Such discussions shall then be referred to the Joint Committee for consideration in revising its report.

Further on the Joint Committee makes the earnest request that every facility be provided by its constituent organization for the fullest consideration of these tentative specifications in order that it may be in a position, as a result of their thorough discussions to reflect in the final specifications the best current practice. We therefore have before us today the progress report, and it should be considered by the Institute in the light of the request that our members should take part in a discussion that will be constructive in its nature and of aid to the committee in drafting its final specifications.

The Advisory Committee of the Institute, in preparing this program for today, recognizing the length of time which would be consumed if we attempted to read the specification section by section, and so discuss it, has attempted to map out a plan under which the discussion will center about the main features of the report; that is, the report has been divided into major sections, which will be discussed separately.

We have organized, during the past fall, four groups of reinforced-concrete contractors, all members of the Institute, to make a study of the report and to be prepared to take part in the discussion here today. This discussion will be led by Mr. Wason of Boston, Mr. Dixon, of New York, Mr. Warren, of Philadelphia, and Mr. Loring, of Cincinnati. We have also arranged with our committees through the chairmen of these committees, to present discussions at the proper part of the program."

DISCUSSION BY LOCAL COMMITTEES.

I. BOSTON, PRESENTED BY L. C. WASON.

At a meeting of the Boston Society of Civil Engineers, called for the special purpose of discussing these Tentative Specifications there were about one hundred present, and about twenty took part in the discussion. The discussion was of a co-operative and helpful nature and not in any sense critical. Those present called attention to various things which they believe could be improved upon. I present herewith part of a digest of this meeting prepared by Prof. Walter C. Voss of Wentworth Institute, Boston.

General.—There is some difference of opinion regarding the advisability of making the Joint Committee report in the nature of a pure specification. L. J. Towne in a paper submitted to the Am. Soc. C. E. meeting in New York, Dec. 8th aptly said:—"The title Specifications for Concrete and Reinforced Concrete is unfortunate, and the form of the old report is much to be preferred, namely ,a set of fundamental principles representing recommended practice for the guidance of the engineer. The term 'Standard Specifications' implies a document which can be incorporated more or less verbatim into working specifications, and which includes all the detailed instructions necessary for the proper execution of the work. The concrete industry, however, is so infinitely varied that the attempt to lay down a set of specific rules to cover the multitude of conditions which arise in practice is likely to result in extended controversy, and if finally adopted, in a weakened authority of the entire report. Whereas, if the report were to be considered as representing the best thought on fundamental principles leaving the details to be worked out by the individual engineer, it would not only be easier to secure agreement on the various provisions included in the report, but the report as adopted would tend to carry greater authority."

C. E. Nichols of Stone & Webster, following the same thought, says, "It ought not to be a specification but a guide to a specification. It isn't and cannot be at the present time." He further states that he wishes to emphasize "the fact that the entire specification should receive our support in as much as we should recognize that it is a step toward better concrete construction." E. S. Learned feels that it is "almost impossible, except as far as principles of design are concerned, to fix anything so inelastic as a standard specification."

If the Joint Committee is able to maintain its position of purposing and constructing a specification, it probably is far better, with our present knowledge of concrete, to confine ourselves to a definition of the result which shall be obtained in each case. The question of preparing a document which can be incorporated into working specifications would be made considerably simpler by so confining its requirements.

L. C. Wason says that "we should endeavor to specify the results to be accomplished and not detailed methods of accomplishing them. If you specify that a concrete floor surface shall not be dusty—that is a result to be accomplished; that is a principle. But to specify that you must not under any circumstances use a drier,—that is a detail. The thing that has built the country up and made it the big, resourceful place that it is, is the fact that it has encouraged initiative in every way—has given a reward for initiative."

It seems that Mr. Wason has struck a key-note in the application of specifications. The positive and limited specification of detail will stop progress. It disallows the use of the peculiar abilities of high grade contractors to produce good results in possibly economical ways. It further hampers the engineer in so arranging his specifications that desirable results may be obtained under special conditions. There are, however, many features about a specification which is prepared with the object, which in the last analysis is the sole criterion of a successfully executed construction, of limiting careless work and of specifying definite results.

One of the foremost of these is called to our attention by H. F. Bryant, of Brookline, Mass., in his statement "that it is a desirable thing that we should have a standard specification which we can refer to in our specification and the contractor won't have much to read because he will have read it once for all."

There is no desire upon the part of the consulting engineer to shift responsibility, as these specifications might lead one to believe. Prof. C. M. Spofford, of the Massachusetts Institute of Technology, frankly states that the engineer wants to assume the responsibility of concrete inspection and of proper construction through superintendence.

To carry out a detailed criticism of every article of the specification would be quite a task. It is hoped, however, that a similar discussion of this document by all constituent societies will approximate such criticism. The major points of difference, however, are well defined, and an attempt is here made to concentrate upon them, with the hope that the Joint Committee may be able to analyze the spirit of the wishes of the profession.

Article 2.—Column Capital:

This definition illustrates the lack of weight given the practical side of the report. Mr. Wason states "the definition is a little ambiguous and ought to specify what "monolithic" means—whether a continuous operation from the bottom of the column to the top surface of the floor, or whether to stop at the bottom of the column head and then begin again. No matter how carefully you pack concrete into a column you are likely to have a settlement of 1/8 to 1/4 in., and if the column happens to be flat-slab design it may be supported and you will get a break in the neck. So that point should be well considered."

Articles 3, 4, and 5.—Quality of Concrete:

The tendency to assist in making this document valuable is illustrated by the suggestions made by L. J. Towne with regard to these paragraphs. He submits the following alternatives:—

"It is suggested that paragraphs 3 and 4 be omitted entirely and that the quality of the concrete be specified under "proportioning" as noted hereafter.

"A separate chapter should be devoted to paragraph 5, Tests of Field Specimens. The A. S. T. M. specification referred to deals only with the technique of making, storing and testing specimens.

"Additional paragraphs should be inserted to cover such items as the following:

1. Provision for preliminary tests prior to starting work including methods of securing fair samples of available fine and coarse aggregates.
2. Number of tests to be made in terms of quantities of concrete poured.
3. Fixing responsibility between the engineer and contractor for the making and curing of specimens and for the cost of testing.

Aggregates:

A clause should be inserted providing that the contractor shall submit to the engineer for his approval samples of the fine and coarse aggregate to be used on the work before the orders for material are placed.

L. C. Wason says that "Section 28 which follows should not be considered in connection with this. It is stated that concrete stored in wet sand develops only 60% of the strength of specimens cooled for four days in the air before testing, so to that extent you penalize the whole structure 25 or 30 percent by making the structure equal only to the weaker test specimen."

Mr. Larned takes the constructive attitude that "it does seem desirable to define more accurately the quality of concrete required, in the matter of consistency in particular, and if the present slump test is the best that can be applied it would be well to use it until something better offers. If unaccompanied with judgment and the experienced eye, with a due appreciation of the variability in sand in the matter of gradation, surface characteristics of the particles, moisture content, and the size and character of the coarse aggregate, it may lead to errors which commonly fall within the field of the human equation."

Article 6.—Portland Cement:

The attempt on the part of the Joint Committee to exact a guarantee from the contractor for the strength of the concrete calls to one's attention the question as to whether the cement manufacturers sell their product on a compression requirement. Such a requirement has been advanced at several stages of the standard cement specifications, but the manufacturer of cement feels that he is not responsible for the peculiar natural notion of his cement with any kind of aggregate, should the requirement be based upon a mortar test. It seems unfair to ask the contractor to assume not only the strength of the aggregate, but also that of the cement and of the resulting mortar.

There is sufficient argument in favor of a compression requirement for concrete, if such a compression test is to be asked at all. Tests of the cement alone will be comparative only and will not involve all the effects which certainly take place in the preparation of mortar or concrete. The question as to where this responsibility shall rest must be decided.

Mr. Larned casts some light upon the attitude of the cement manufacturer toward this subject when he says: "The cement manufacturers sell cement today which is guaranteed to meet the requirements of the standard specification of the American Society for Testing Materials. At the present time they have only a tensile requirement. There is a tentative specification providing for compression tests and requirements, but this has never been incorporated in the standard specification. When it is, it is assumed that it will be reasonable, and that cement manufacturers will probably guarantee their product under this requirement also, but they only guarantee cement, and in my judgment they will never guarantee the sand, the coarse aggregate, or the personal equation in making concrete."

Article 11.—Mortar Strength Test:

The comparison of tests of mortars with the standard Ottawa sand mortar has been considered meaningless and without value by many engineers. The imposition of such a comparative check upon the fine aggregate works hardships upon many contractors who are forced to import the Ottawa sand from long distances in order to comply with the specification. When one considers that the cement is acceptable under the specification if showing a tensile strength of 200 lb. per sq. in., and that some natural sands showing a strength of 246 lb. per sq. in. have been condemned because they did not show 100% of the strength of the Ottawa, one wonders if the wise thing has been imposed. Certainly the practical, sensible course is not being pursued.

Mr. Larned says: "Should we not rather fix a definite strength for natural sand or fine aggregate in pounds per square inch without further reference to Ottawa sand, which is only used for laboratory purposes."

The lack of practical intent of the specifications is further evidenced by the fact that "generally speaking it is not possible to test concrete cylinders on the site of the work. It requires testing machines which are found in the laboratories of the country but are often not found anywhere near the work." This often necessitates the shipping of cylinders over long distances.

Prof. C. M. Spofford says regarding the Springfield Bridge tests:—"We have been making tests of concrete used at Springfield on the bridge there. In order to do this we have had to ship them to Boston and do the testing at Technology—at least we found that the most convenient place to make these tests. In order to get them up here to make the seven day tests they had to be shipped when they were not very thoroughly set and it is quite possible that the jarring of the concrete during shipment would have a material effect on the strength of the cylinders. I think this feature has not been considered."

Articles 16 and 17.—Rubble and Cyclopean Aggregate:

These articles again point to the lack of detail which a *real* specification should possess. It is suggested from time to time that the engineer may add these, but why call the whole a specification if it is not?

Article 26.—Unit of Measure:

"The unit of measure appears to have grown since the work of the original Joint Committee, 94 lb. of cement or one bag is to be considered as one cubic foot formerly, largely for purposes of convenience in gaging a barrel of cement, or four bags weighing 380 lb. was called 3.8 cu. ft. This was quite a growth in view of the specification then existing, where based upon the actual volume measurement of the commercial cement barrel, it was found that the actual content between beads in the case of the largest barrel found among 8 or 10 brands, including foreign brands, was less than 3.5 cu. ft. and many important engineering undertakings were carried through where the unit of measure was called 3.5 to 3.6 cu. ft. per barrel. The present requirements is a liberal move in favor of the contractor, but it can hardly be interpreted as an added factor of safety in the strength requirements of concrete."—L. C. Wason.

Article 27.—Method of Measuring:

This article "specifies two alternatives for measuring constituents—by volume or by weight. In a *general specification* this is rather unwise. It would lead to controversy on the work, speaking from the business end rather than the technical end, and there are now only inadequate methods of weighing concrete in quantities, as would have to be done on large work. It is fairly easy to measure it by volume. It would add to the expense of executing the work to weigh your aggregates."—L. C. Wason.

Article 28.—Proportions:

Never has a tentative specification advocated so radical a departure from existing practice. One wonders if someone has suddenly developed the heretofore unknown trick of concrete mixing, which because of its very satisfactory disposition of natural inequalities and the human equation makes a uniform reacting composition. It is probably the general opinion of engineers that no such trick has been discovered. The general opinion which exists is that the Committee has overstepped its interpretation of present conditions. Nevertheless we are anxious to travel in this very definite direction.

L. J. Towne says: "The proposal to proportion concrete to secure definite strength based on laboratory tests of compression cylinders represents a distinct advantage in the art which will tend to economy of materials and a better class of concrete construction. It should therefore receive the thoughtful consideration of every engineer. One can agree with this principle, however, and at the same time hold grave doubts whether the time has yet come when concrete can be purchased on the basis of strength in the same way that structural steel is now purchased.

"It is true that thousands of laboratory tests have apparently established with reasonable accuracy the relation between the component parts of the concrete including water and the resulting strength of the specimen. On the other hand, the question of the relation between the strength of specimens made in the field and the strength of laboratory specimens from the same aggregates has had comparatively little attention. The writer had occasion recently to compare a series of field tests of compression cylinders made under working conditions in which the water content and "slump" had been carefully recorded, with the predicted values in the table printed with the Joint Committee's Progress Report. The results showed no relation whatever between the actual and the predicted strength. Probably if all the variable factors were known including the personal equation of the man making the specimens, the discrepancies might be satisfactorily explained, but the experience brought forcibly to mind the fact that there is much ground to be covered before we may confidently predetermine the strength of concrete under conditions existing in the field.

It seems especially unwise in the present state of the art to fix the responsibility for securing concrete of a definite strength on the contractor. The responsibility for determining the proper proportions of cement aggregate and water should rest squarely on the engineer.

When the contractor has followed the engineer's directions in this regard as well as his specifications for time of mixing and workmanship in general, he has gone as far as he can be expected to go and should not be held responsible for the strength of the concrete, except where defective work can be traced to his failure to conform to the engineer's specifications for either material or workmanship. Any other policy will result in endless confusion in the enforcement of the specification and will work to the disadvantage of the honest contractor and in favor of the less scrupulous contractor who is willing to take a long chance."

Some very direct arguments in opposition to this article are advanced by Messrs. L. C. Wason, A. B. MacMillan and Prof. C. M. Spofford in the following extracts:—

(a) "Looking at it from a business point of view, if the contractor is going to play safe, obliges him to get his equipment set up so that the materials can be operated on a commercial basis, as he would in the work. Take his machinery, materials, and shut down the work for 28 days to make the test,—imagine what that would be. Yet that is the only way you can possibly be safe, and if you don't do that you might put up two stories of a building before your tests are made and find by your tests that the work is below standard and you are liable to a loss. The specifications nowhere call for a factor of safety or for tests of the building—only for laboratory tests, and the whole work is to be judged by laboratory tests. There is no alternative. That, from a business point of view, is very bad.

(b) "The second alternative eliminates one of these bad features. It allows the contractor to proportion his materials to give a certain strength. A footnote says prepare your contract so that if you have a lawsuit you will know what procedure to follow. They recognize that trouble might arise, but don't try to eliminate it. Now a lot of contractors are able to make tests and proportion materials correctly, but a great many more of them are not. Therefore it is better to put the responsibility on the engineer, where it belongs. Let him assume the responsibility of design and strength and let the contractor merely execute the work under his supervision.

"The third method of specifying definite proportions is all right, but it refers to Table 4 which gives elaborate percentages of different degrees of strength and size of materials. It would be much better to eliminate that table and put the responsibility upon the engineer to select the proportions which he wants, and then if that affects the cost, let an adjustment be made. Do not let the contractor assume part of the responsibility which belongs to the engineer."—L. C. Wason.

"One thing this article does not mention is the possible burden on the contractor of agreeing to furnish concrete of a given strength before he has the job,—in other words, some 28 days before the contract has been obtained he must visit the gravel pits and make various tests and determine by analysis that the gravel, sand, and cement which he proposes to use are correct to give concrete of the strength that is required. We have found in actual practice on large work that it is frequently impossible to get sufficient gravel from any one pit or stone from any one quarry. Suppose that instead of determining the quality of the sand in one pit you have to go to several. The results are obviously uncertain: very far from anything you can predict in advance."—A. B. MacMillan, Aberthaw Construction Co.

"The first one of these is not that we as engineers would care to use. We wish to take the responsibility for the mixture, for the strength of the concrete, rather than to allow the contractor to take it. If the contractor's results have got to be determined by 28 day tests it seems to me personally a very difficult method of procedure. He may have the building up several stories before the ground story can have been tested. What are we going to do if we find that it is not strong enough? Mighty few engineers could induce the contractor to pull down the building and start over again and I think it would be difficult to produce the force to require him to. It seems highly impractical. The last method of proportioning seems to conform to present practice if we omit the reference to Table 4 and possibly the rest of the clause. (I am not sure about that.) I think the Committee should reach a decision as to which one of these it wishes to recommend, if the report is to be of value. I am as much in sympathy with the attempt to obtain in concrete a material that can be ordered by strength, as anyone. If we could specify the strength of concrete as we specify the strength of steel it would be a fine thing, but I see no means of doing it at the state of the art. We can buy steel with certainty. It can be tested while it is being made and if it is not up to the standard the steel can be rejected and used on some other job, but with concrete it is another matter. When it is made something has to be done with it. It must be used on the job, and I don't think we have yet reached the point where we can hope to write such a specification and enforce such requirements."—Prof. C. M. Spofford.

The general impression that this article is a step in the right direction is well stated by C. E. Nichols, of Stone & Webster—"I should like to emphasize that, not only is Article 28 a step in the right direction, but that I think we ought to recognize it and encourage it. I don't think we ought to say that we can't specify concrete of a given strength and expect to get it. We all hope to reach or approximate that sooner or later. For that reason I think we ought to encourage it."

L. J. Towne constructively criticizes the article by supplying the following substitutes:—

"The proportion of cement, water, and aggregates, and the desired strength shall be as follows:

Class	Portland Cement	Fine Aggregate	Coarse Aggregate	Consistency by slump test	Desired Strength at 28 days
A	1	....	....	....	....
B	1	....	....	....	....
C	1	....	....	....	....
D	1	....	....	....	....

"The quantity of water used shall be such as to produce concrete of the consistency specified above as determined by tests. After the engineer has approved the fine and coarse aggregate to be used for the work, and before any concrete is poured, he will advise the contractor whether any changes are desired in the proportions given in the above table in order to obtain the desired strength. The engineer may also at any time during the progress of the work make such changes in the proportions of water, fine and coarse aggregate as are necessary to obtain concrete of the desired strength as determined by tests. Where such change involves an increase or a decrease in the cost of the concrete materials, the price for the work shall be adjusted as provided in the contract."

Article 29.—Consistency:

There is no question but what this feature of concrete mixing is of very great importance. Something should be done to limit the careless use of water. The Committee loses sight of the fact that it delegates to the engineer full responsibility for this feature, while it holds the contractor responsible for the strength of the resulting concrete which they themselves tie up inseparably to the water-cement ratio. Divided responsibility of this kind will lead nowhere.

Article 30.—Machine Mixing:

This article "specifies equipment which I think does not now exist. You cannot specify a definite water-cement ratio and expect to live up to it. If you use a locked measuring device you will get all kinds of mixtures. Twenty-two quarts of water with a two bag mix,—I think that is about the proportion,—and see that an intelligent man is on the mixer. But it varies from two quarts to four quarts to get the same apparent consistency and I will defy anyone to carry out a construction job and make a slump test of every batch. That is the only way he can see what he is doing. He has got to use his judgment. Make adjustments from batch to batch, in order to get the consistency desired."—H. F. Bryant.

"There are also methods specified which effect concrete machinery men. "The water shall be measured by an automatic device," and later on it specifies that the device shall be kept locked and not under the control of the contractor or the workmen. The moisture in aggregates will vary from day to day and perhaps on the same day, and it may not be convenient for the engineer to change the quantities of water to meet the emergency."—L. C. Wason.

What would be more foolhardy than to tie the contractor down to conflicting conditions. It seems that it is well to take the spirit of the above criticisms into consideration.

Article 31.—Time of Mixing:

The unnecessary increase of mixing time will show up in the cost of a building. It is unquestionably well to provide a minimum, but it certainly is unwise to expect a time which is of no great advantage. A. C. Tozzer approaches this item with the comment: "The time of mixing comes up very frequently in specifications and on actual work.—'The mixing of each batch shall continue not less than 1½ min. after all the materials are in the mixer.' It has been brought out by test that increasing the time from ½ to 1 minute gives only a 10% increase in strength and from 1 to 1½ minutes only 2% additional. These were tests conducted at Purdue University."

Article 37.—Spouting:

Mr. Larned says: "Is it expected for a moment that a practically continuous flow in the spout could ever be maintained? The provision for discharging into a hopper is a timely one in view of the foregoing requirement."

Article 40.—Protection:

"This paragraph is "the only thing on protection of surfaces:—just two lines, "Exposed surfaces of concrete subjected to premature drying shall be kept thoroughly wetted for a period of at least 7 days." If you are going to use these specifications in real earnest you must take them at their face value, on what they say, not what the Committee intended to say. That literally means that every exposed surface, walls, ceilings, floors, and columns must be protected by being kept wet for a period of at least seven days, and you all know what an impossibility that is, and how unnecessary. Moreover there is no description of what "premature drying" means. That assumes "premature" is a correct term, but we are not told what it is."—L. C. Wason.

Articles 41, 46, 48, and 54, etc.—

The outstanding conglomerate nature of the specifications with regard to principle and detail is again called to our attention by these articles. If detail must exist, it should be left to the engineer and contractor to work out in the light of the particular conditions.

Article 67.—Fire Protection:

Next to Article 28, with its tendency toward increased cost of construction because of "playing safe," this clause discards or tends to discard one of our very plentiful and heretofore useful aggregates,—gravel.

Mr. Wason approaches this article by saying that the "extra protection under certain conditions which practically eliminate the use of quartz or gravel concrete. Oftentimes this is the only available material or the cheapest material and should be permitted and not absolutely excluded. We will admit that in certain places where there is such extreme fire hazard and extra risk it is wise to use better aggregate than quartz, but this wholesale exclusion is an unnecessary expense to the owner."

Mr. Tozzer remarks that "the provisions for fire protection would throw out, of course, the quartz gravel. Consider the case of a spiral column where gravel was used:—that would mean that the diameter of your column would be increased 2 in. and you would have to place (outside of the spiral) a wire mesh. We all know how hard it is to get the concrete around a spiral, but when we have to get it around a wire mesh also, I am afraid the amount of water would be in excess of what is allowed by the slump test."

Mr. Bryant further remarks about this article "The requirement of 1 in. cover for fire protection for the steel in flat slabs is quite onerous. There is a big difference between 1 in. and ¾ in. I think we all do. It is a question of compromise and that 1 inch means weight. It means a very large percentage. It runs into money, and we don't get 1 in. now.

What is the use of providing it if we aren't going to get it. I think nobody will object to ¾in. and I hope this won't get into print as it now stands in the Committee's specification. The additional inch for gravel or stone, I think is grotesque. I believe there is no question that trap rock is much better protection, but personally I don't know of any severe injury to flat slab surface from fire where the injury was due to the use of quartz stone or gravel. If there is, it is much more than counterbalanced by the convenience and safety obtained by using a lighter slab."

Article 75.—Expansion Joints in Long Buildings:

This matter needs considerable study. There are various differences of opinion and it may be possible to extend the 200 ft. limit with proper care in temperature reinforcement. It certainly is wrong to fix a single limit. Perhaps 200-300 ft. would be better.

Articles 79-80.—Waterproofing:

There is no question but what integral components have been used with success, and there are cases where membrane waterproofing has failed to produce good results. As Mr. Towne says:—"It is unfair to rule either type of waterproofing out entirely."

Article 82.—Oilproofing:

If the Committee indulges in details, why not here? If it intends to base its specifications upon results, why not do that clearly? As this article now stands it is not at all explicit in either direction. The fact that it is to be rested with the engineer indicates that at times the Committee intended the more reasonable type of specification.

Article 96.—Curing:

This article is significant of at least an unintentional ignorance of the practical phases of road construction. Mr. Larned elaborates upon this phase:—"21 days is 14 days in excess of the present requirements of practically all the highway associations in the country. Just what they mean by 'pavement' I don't know. I would assume it might include sidewalks, in which case you would be excluded from the use of a sidewalk for at least 21 days, plus the time of construction. In the case of a highway I recognize the difficulty of taking care of traffic during construction and of throwing the road open as soon as possible. Yet the contract calls for two weeks at least before the concrete could be put in use, and when it was put in use a 40 ton truck with iron wheels was to go over it. What can you do to maintain a detour for 21 days? The last stretch of the work you put down is the controlling factor. It may take two or three months to build a three mile stretch. What are you to do,—wait for the job to be finished and then 21 days longer before you use it?"

Section XI. Design

In this phase of the report the engineer is probably the most interested party. In general these articles are as before. They form a very valuable part of a "recommended practice" report, but as specifications material is probably superfluous in the main. One of the changes, however, which does concern even the public is the penalizing of lighter columns. If they are unsafe then we are in danger of concrete failures for some time to come.

The following extracts will serve to call attention to this fact and its far reaching effect on construction costs at a time when they are already high.

"The design recommended by this Joint Committee was compared with the New York Code and it was found that a whole building would cost 11% more by the Joint Committee specifications than by the New York Code. The columns would cost 30% more. If the existing methods of building codes have been satisfactory and have given good results there is no need of penalizing construction by putting up a structure which is better than is needed. Your engineer specifies what he wants, what is good enough. There is no need to compel it to be 11% more expensive."—L. C. Wason.

"In the early years of concrete building where everything was hit or miss amongst a great many contractors, there were a number of failures, which today we hear nothing of. Today we immediately put a failure down to carelessness or inefficiency. That brings out the fact in our minds very clearly that this increased cost which must come by more rigid requirements is something which would be a very serious blow to the economic side of the concrete industry."—A. C. Tozzer.

Prof. Spofford also finds the following fault with Article 197:—"I notice, for example, the allowable unit stress in steel which says that rail steel bars shall be figured at 16,000 lb. per sq. in. That refers to re-rolled steel. That, of course, arouses at once a good deal of antagonism among makers of re-rolled steel. I don't think there is enough information on hand to warrant reaching an opinion that re-rolled steel is not perhaps as good as the intermediate or hard grade of billet steel. I think the Committee should study that point further before issuing a definite specification for it. Also the strength of concrete, which goes up to 800 lb. when we are only accustomed to using 650 lb. I am not certain that is a proper change to make at this time. We have had a good deal of experience in testing concrete cylinders and I am quite in agreement with the statement of how variable the results are in tests on cylinders taken on concrete on the work. I suppose we have tested many hundreds in the last few years and the results are certainly quite variable."

E. J. Moore.—Could we get Mr. Wason's recommendations for the time of mixing in place of that minute and a half, whether it is one minute or three-quarters of a minute? Mr. Moore.

L. C. Wason.—I should say not less than half a minute and not necessarily more than three-quarters of a minute would be sufficient. Mr. Wason.

Sanford E. Thompson.—Mr. Wason has referred to Article 28, on which there has been a great deal of comment in various quarters. I would like to ask him if he has prepared any constructive suggestion as to a substitute for that article? I know as a member of the Joint Committee, that we are anxious to make that article so that it is practical and yet covers the theoretical points. Mr. Thompson.

Mr. Wason.—I have not put it in exact words, but it is the expectation that groups of engineers who are discussing it will get it into that shape pretty soon. It is my personal belief that the first two alternatives should be eliminated altogether; that the third should be modified by cutting out the reference to Table IV which specifies proportions or sizes which do not universally exist throughout the country, and let the engineer, from a test of the actual materials, determine what you can do under the circumstances of the particular materials he has at hand, that is, specify proportions, in that third alternative, from his own tests. Mr. Wason.

Duff A. Abrams.—I would like to have Mr. Wason indicate just how the engineer should determine this matter of proportions without using the table and following his own suggestions: Should the engineer simply take the aggregate as it is and determine what the proportion should be? Mr. Abrams.

Mr. Wason.—If your materials are of the sizes which fit your tables, use them: if they do not fit the table, determine for the particular case in mind, what they should be. Mr. Wason.

Mr. Abrams.—That is just the point I would like to have explained. How should he do that? Mr. Abrams.

Mr. Wason.—By taking materials and making tests. Take the materials as soon as the contract is awarded. Probably you will have your test done before you get very far in your work, before you get your foundation done. It is in the superstructure that exact knowledge of strength is required; there is where it is important. I think you would have time to make your tests as soon as they awarded the contract and before they started actual work on construction. Mr. Wason.

Mr. Abrams.—I understood that was one of your objections to the report, that the job would have to be held up twenty-eight days after everything was all set, waiting for the tests to be completed. Mr. Abrams.

Mr. Wason.—I did say so. As I understand this first alternative paragraph, you take tests at regular intervals through the work. My suggestion is that you take them at the very beginning and that will be all. I would like to state one thing I did not say in the beginning Mr. Wason.

Mr. Wason. of my remarks, that there is no reference whatever to factors of safety, nor to actual tests of the structure. You depend entirely on laboratory tests for your finished structure; if laboratory tests fail to measure up to your requirements, the structure is weak; the natural way then would be to make some tests of the structure, but the specifications do not permit that alternative and make no reference whatever to a factor of safety as a basis for determining whether the structure is strong enough or not. The only alternative is to make an adjustment with the owner or to resort to the law courts.

Mr. Humphrey. Richard L. Humphrey.—As I understand Mr. Wason's answer to Prof. Abrams, when you went on the job you would propose for the contractor to take whatever materials were available and make the tests; would you carry that same proportion throughout the entire structure?

Mr. Wason. Mr. Wason.—If the materials are uniform one set of tests ought to be enough. If they come from different sources, test each one. Sometimes you cannot get all the materials from one source, and in that case you would make another test. If it is evident the materials lack uniformity, make tests enough to keep the desired results under control.

Mr. Humphrey. Mr. Humphrey.—What would you do in that case; hold the construction up?

Mr. Wason. Mr. Wason.—That would be the safest thing to do, but it is not the usual thing to do, because an engineer has sufficient experience to determine whether the new materials are equal in strength to the old quite promptly.

Mr. Humphrey. Mr. Humphrey.—Assuming that the materials all come from the same source, do you think a single test at the beginning of the operation would be sufficient?

Mr. Wason. Mr. Wason.—Probably, yes.

Mr. Lindau. A. E. Lindau.—I should like to ask Mr. Wason to make some recommendation or express his opinion as to the character of the load tests that would determine the success of the work. That has been a mooted question, how they should be carried out and what should be the criterion of a successful job as measured by the load test.

Mr. Wason. Mr. Wason.—If you were building a structure with a given floor load, like 200 lb. per sq. ft. and have reason to believe that your materials may be a little weak, I think that if it is tested to twice that load and the deflections are within limits that experience proves to be safe, that the floor is all right. It is my practice to put on twice the load which the structure is designed to carry, in making a load test.

Mr. Lindau. Mr. Lindau.—I would like to ask still further whether you would base the success of the load test on a deflection measurement or recovery? Your modulus may be 4,000,000 or 2,000,000, and your deflections may vary with the modulus; it depends on whether you want to use a deflection measurement or recovery measurement as the basis.

Mr. Wason. Mr. Wason.—I take both into consideration, and by consideration of all the facts available, determine whether the floor is strong enough to put into actual service.

Mr. Lindau.

Mr. Lindau.—I would like to ask if you would think that the provision used, I think, in the Chicago Ordnance, of 80% recovery, a satisfactory criterion for a job.

Mr. Wason.

Mr. Wason.—Yes, I think it would be. I know of a number of cases in my own work where I got 100% recovery; if you get 80% recovery, I think that is satisfactory.

Mr. Lindau.

Mr. Lindau.—I would like to ask Mr. Wason if he thinks that 30 days' load test would be satisfactory from a contractor's standpoint, or would impose an unnecessary limitation?

Mr. Wason.

Mr. Wason.—If a contractor has done work which is, say, open to reasonable doubt as to strength, he has got to stand to have a proper test made, whether it is satisfactory to him or not. It is his fault, and therefore I do not think you need to consider him, whether a test is made one month or one day, the owner's interest then is at stake and the owner should do what is for his own best interest, not the contractor's.

Mr. Brown.

H. W. Brown.—Suppose there is no fault evident at all, you are simply trying to find out whether the construction is proper. You say it is the contractor's fault; he may be at fault, but there may not be any evidence of the fault; you are trying to find out whether it is wrong or not.

Mr. Wason.

Mr. Wason.—If there is a reasonable doubt, you have got to find out. I think it is up to the contractor to prove that his work is all right.

Mr. Humphrey.

Mr. Humphrey.—I would like to ask Mr. Wason whether he thinks the test of a floor would be an indication of the value of the column in the lower stories? Would you load more than one floor or an entire floor, or how would you get a test on your column?

Mr. Wason.

Mr. Wason.—If you have doubts about the columns in your lower stories, put sufficient tests on the building to prove whether those lower columns are right or not. That might require loading several floors or all the floors around one particular column to prove whether the first story is right or wrong. I think the engineer should use his own best judgment to determine beyond a reasonable doubt, if those columns are satisfactory.

Mr. Humphrey.

Mr. Humphrey.—I would like to ask Mr. Wason if he cares to say whether he thinks the cost of those tests should be borne by the owner or by the contractor.

Mr. Wason.

Mr. Wason.—That would depend somewhat upon conditions. The contractor is liable to be at fault, and if his work shows a reasonable doubt so that it requires a test, I think it should be at his expense to prove that it is satisfactory.

Mr. Wight.

Frank C. Wight.—Can Mr. Wason tell us how he would establish a criterion of this "doubt" he speaks so much about?

Mr. Wason.

Mr. Wason.—I do not think you can; it is usually an argument between the engineer and the contractor, or the owner may come in too. I think it is always a special case.

II. NEW YORK, PRESENTED BY DEFOREST H. DIXON.

At a meeting of New York Contractors interested in concrete and reinforced concrete construction held on September 13, 1921, a Committee was appointed consisting of J. G. Ahlers, of Barney-Ahlers Construction Corpn.; G. Edward Escher, of White Construction Co.; N. M. Loney, of Thompson-Starrett Co.; M. M. Upson, of Raymond Concrete Pile Co.; and D. H. Dixon, of Turner Construction Co. This committee was requested to make a study of the Progress Report of the Joint Committee and to arrange, so far as possible, for the testing out in actual practice of the provisions of that report having to do with proportioning, control of water, testing af concrete, etc.

The Committee worked along the following lines:

1—Prepared the following memorandums to assist contractors in taking up the study of the grading and proportioning of aggregates and the control of water cement ratio—
  (a) The design of concrete mixtures.
  (b) Suggestions for job tests of concrete.
  (c) Water measuring devices.

2—Instituted trials on jobs of the methods suggested in the tentative specifications.

3—Conferred with experts in the designing of concrete mixtures and in the testing of concrete.

4—Studied the available technical literature on the subjects under examination.

5—Conducted experiments on the water content of aggregates, strengths of different cements when tested at 28 days, etc.

6—Prepared comparative estimates of cost when designing under the New York Building Code and under the tentative specifications.

The committee submitted its report on Nov. 30, 1921, in the form of a general report containing a brief statement of its comments on the various sections of the Tentative Specifications, accompanied by supplemental reports on some of the more important matters requiring study in connection with a consideration of the provisions of the Tentative Specifications. These supplemental reports were as follows:

1—Mr. Loney submitted a supplemental report on comparative costs under the Tentative Specifications and the New York Code with comments on reasons for the differences in cost.

2—Mr. Upson outlined the new relationship that would be established between the Owner, Engineer or Architect, and Contractor, by the provisions of Section 28, etc. and explained the possible developments that might result from this new relationship.

3—Mr. Dixon submitted the supplementary report giving the technical reasons why concrete cannot be sold by strength measured by cylinder tests at 28 days.

The speaker will try to summarize these reports as briefly as possible. He regrets that the limitation of time available permits only a brief reference to the effect of the Tentative Specifications in establishing new relationships between the Owner, Engineer or Architect, and the Contractor. This is a very important matter. It is to be hoped that Mr. Upson's study of this question will become available in printed form through the discussion at the American Society of Civil Engineers.

COSTS UNDER THE TENTATIVE SPECIFICATION COMPARED WITH COSTS UNDER THE BUILDING CODE OF THE CITY OF NEW YORK.

After giving the matter considerable thought the committee decided that it was doubtful if reliable results could be obtained by trying to compare the costs of the various parts of a building, such as floor slab, interior columns, footings, etc. considered each as a separate item, because to design to cheapen one item might increase the cost of other items. For example, when designing for light floor loads under the Tentative Specification the use of 1500-lb. concrete for the flat slab might save money in the construction of the floor slab even though more concrete was used than would be necessary if 2000-lb. concrete was used, but the greater thickness of concrete required with 1500-lb. concrete would load up the columns and footings and increase the cost of those items. There were other secondary effects for which it was thought it would be even more difficult to make proper allowance in a final analysis.

The committee therefore decided to have its comparisons made through estimates which in each case would cover the entire skeleton of a building, including footings, and columns, and floors, and roof. It was appreciated that to be of value the results had to be worked out as accurately as possible, and that each result should, if possible, be the average of two or more independent estimates.

The committee drew up a specification for the reinforced-concrete skeleton of a factory building six stories high, with no basement, and having outside dimensions 80 x 300 ft. This specification contained the necessary information for an accurate estimate, such as column spacing, bearing power of the soil, etc., except no structural details were shown, and the engineering and estimating departments of each company taking part in the work of the committee was asked to design the structural members of the building as cheaply as possible when figuring under either of the two codes.

The prices for cement, sand, gravel, steel reinforcement in place, spiral wire in place, and labor per yard of concrete, including installation, were all set up in the specifications.

After studying Table IV of the report, which seemed to give mixes consistent with the results that could be expected on wet cylinders, the mix of concrete necessary in order to obtain 1500 lb., 2000 lb., and 2500 lb. concrete under the Tentative Specifications was given as follows:

1500 lb. per sq. in.......	1 cement,	2.4 sand,	3.5 gravel
2000 " " " "......	1 "	1.8 "	2.9 "
2500 " " " "......	1 "	1.3 "	2.4 "

Under the New York Code 1 : 2 : 4 concrete, and 1 : 1½ : 3 concrete was used in accordance with the provisions of that code.

The quantities of cement, sand, and gravel per yard of concrete were in each case taken from the Thatcher Tables and given for each of the mixtures.

As it was desired to find out the effect of using concrete of differing strengths when designing under the Tentative Specifications, separate estimates were requested under the Tentative Specifications using 1500-lb. concrete, and using 2000-lb. concrete, and using 2500-lb. concrete for all portions of the skeleton, unless in the judgment of the designer by so doing the columns became larger than is allowed by good practice. To properly control the size of the columns it was provided in the specification that before starting to estimate the skeleton for a given floor loading the engineer was requested to decide upon the largest column that would be considered to comply with ordinary good practice in the experience of his company, and he was then requested not to use a larger column in figuring the estimates for that floor load under the Tentative Specifications or under the New York Code.

In order to be able to obtain data as to the costs of the various portions of the building a tabular form was prepared and furnished to the engineers and estimators, so that when the estimates were given to the committee they showed the required quantity of steel, the required quantity of concrete, and the total cost for each of the following portions of the skeleton:

1—Floors and Roof.
2—Exterior wall lintel beams.
3—Exterior columns including brackets.
4—Interior columns including capitals.
5—Exterior column pedestals.
6—Exterior footings.
7—Interior footings.

The cost of the forms was omitted as having a small effect on the comparisons we wished to make.

In order to get comparisons with different floor loadings estimates of the cost of the skeleton were made for both flat-slab and beam-and-girder construction, using live-loads of 75 lb., 150 lb., 300 lb., and 600 lb. per square foot of floor.

The work of designing and estimating was divided up among the companies represented on the committee

Barney-Ahlers Construction Co.,
Raymond Concrete Pile Company,
Thompson-Starrett Co.,
White Construction Company,
Turner Construction Company.

Three companies figured the 150-lb. series as the most important, the 300-lb. series was figured by one company, the 600-lb. series partly by one company and partly by two companies, and the 75-lb. series by two companies. Altogether fifty-six estimates of the building were made, each divided up to show separately the cost of the floors and roof, interior columns, etc., as previously described.

When the estimates were received considerable variation was found in the results on footing design, due in large measure to the fact that some companies design sloping footings and others step footings. The cost of the footings is however only about 10% of the cost of the skeleton so that any variation in this item has a small effect on the general results, especially as each company would presumably follow the same method in making its design for both the New York Code and the Tentative Specifications.

The estimates when received were tabulated by the committee and show the following results. In each case the percentage in the following table is the increase in cost of the reinforced-concrete skeleton (exclusive of the forms) when designed under the Tentative Specifications above its cost if designed under the New York City Code, unless a minus sign is prefixed to the percentage:

Flat-Slab

Loading.	Tentative Specifications.			New York Code.
	2500-lb.	2000-lb.	1500-lb.	
75 lb.	15¾	9	5¾	0
150 lb.	13	9¾	9¾	0
300 lb.	13	8	11	0
600 lb.	13½	9	11	0

Beam & Girder

Loading.	Tentative Specifications.			New York Code.
	2500-lb.	2000-lb.	1500-lb.	
75 lb.	+5½	+3	+3¾	0
150 lb.	+4¾	+4¾	+4¾	0
300 lb.	+4	0	+1	0
600 lb.	−1¼	− ¾	+4	0

Flat-Slab Design.—It will be noted that the average for the twelve cases reported under flat slab is 10¾%. It will also be noted that for the 150-lb. load, which is the loading used in the majority of concrete buildings, that the increased cost due to the use of the Tentative Specifications with the most economical design possible under these specifications is 9¾%.

The items entering into this increase of 9¾% are shown in the following statement, where the figures given show the effect of the various items listed on the total cost of the skeleton of the building (omitting forms) expressed as percentages above the cost of the skeleton if designed under the New York Code:

Floors and Roof Flat Slab

	Per Cent	
1—Richer mix required	4.7	
2—More concrete required	1.5	
3—Less steel because steel figured at 18,000 lb. instead of 16,000 lb.	2.7	
4—More steel because moments are increased one-sixteenth above New York Code	1.3	
Total for Floors and Roof		4.8

Interior Columns

5—Increased cost including concrete and steel		3.4

Exterior Columns and Pedestals

	Per Cent	
6—Richer mix of concrete required	0.8	
7—More concrete required	0.1	
8—More steel required	0.2	
Total for Exterior Columns and Pedestals		1.1

Footings

9—Less concrete but more steel		0.5
		9.8

The principal reasons for these differences are as follows:

(1)—A new measure for the strength of concrete, namely the strength of cylinders tested wet at 28 days is set up in the Tentative Specifications. This requires the use of richer mixes of concrete under the Tentative Specifications.

(2)—The moment coefficients of the Joint Committee for floor slabs require that one-sixteenth more stress be provided for by means of concrete and steel than is required by the moment coefficients of the New York Code.

(3)—The design of the interior columns is entirely changed. Under the Joint Committee Tentative Specifications no value is given to spiral reinforcement in figuring the carrying capacity of the column.

(4)—The cost under the Tentative Specifications is reduced by the fact that steel is figured at 18,000 lb. per sq. in., while under the New York Code only 16,000 lb. is allowed. If the New York Code should be changed to allow 18,000 lb. then the increased cost under the Tentative Specifications would become some 3½% greater, or about 13% instead of 9¾% when considering the most economical design for the usual loading.

Beam and Girder.—The comparative estimates for beam and girder construction show that the requirements of the Tentative Specifications will increase the cost of the skeleton frame for such a building about 3% if the average is taken for the twelve conditions figured. For the more common floor loading of 150 lb. the increase by the use of the most economical design under the Tentative Specifications is about 4¾%.

It is very disappointing not to find a reduction in the cost of beam and girder design under the Tentative Specifications, because it has been the consensus of opinion for a long time that the factor of safety now present in our beam-and-girder construction is larger than required by conservative design.

The Tentative Specifications does reduce the moment coefficients, in figuring beams and girders, etc., and figures steel at 18,000 lb. instead of 16,000 lb., but the saving in the floor construction effected by these changes is wiped out by the increased cost of the richer mix of concrete required by the new standard set up for the strength of concrete. As the Tentative Specifications effects no saving in the floor construction the increased cost of column construction, etc., results in the 4¾% increase when the skeleton of the beam and girder building is considered as a whole.

OTHER ITEMS OF INCREASED COST.

In comparing the costs due to differences in design under the two codes, no allowance was made to cover the following items of cost that would be required under the Tentative Specifications if left in its present form:

Item 1.—In our comparative estimates we assumed that quartz gravel concrete could be used when we figured under the Tentative Specifications. If paragraph 67 was continued in force this would be impossible. In New York the substitution of other aggregate would cost about 47% more figured on the cost of the skeleton without forms.

In other cities and in New York when the Hudson River is frozen up the increase in cost would sometimes be very much more serious because of the difficulty in obtaining aggregate other than quartz.

Item 2.—The cost of an expansion joint was not figured in our estimate, but the expansion joint would be definitely required under the Tentative Specifications as they now stand. Such a joint would cost about 3.1% of the cost of the skeleton exclusive of forms, not allowing for the use of plates on the exterior columns to cover the joint which would be used in many cases and increase the cost still more.

Item 3.—Dusted-on finish is used quite frequently. For buildings with monolithic finish the Tentative Specifications would prohibit the dusted-on method and increase the cost for these buildings about 2.4% of the cost of the skeleton, exclusive of forms.

Item 4.—No allowance was made in our estimates under the Tentative Specifications to take care of increasing the time of mixing to 1½ minutes, nor for the additional plant required in certain cases if such a requirement was adopted.

Item 5.—No allowance was made for the cost of attempting to control the water content in the aggregate in order to keep a uniform water-cement ratio in the concrete. This is a very difficult matter and might have a serious effect on costs through the expense of methods used to try to keep the moisture content of the aggregate uniform, or failing in this attempt the extra cost of having to use a mix rich enough to take care of the wetter batches sometimes mixed, through the presence of more moisture in the aggregate.

Item 6.—If spouting systems as at present operated were not allowed, then the cost of all buildings where it is now possible to spout to advantage would be materially increased if built under the Tentative Specifications.

Item 7.—No allowance has been made for the additional labor because of the greater difficulty in placing a drier mix, or for conducting the tests, etc.

If the Tentative Specifications is adopted as at present written the increases due to *Item 1—No Quartz,* and *Item 2—Expansion Joints,* will be absolutely required. This will bring the increased cost of the skeleton for 150-lb. flat-slab, under the Tentative Specifications, up to the following:

Increased cost due to design	9.75
Impossible to use quartz	4.7
Expansion joint required	3.1

These three allowances add up to 17.5%, to which will have to be added 3% more in buildings where dusted on finish is now used, and probably a considerable additional allowance to take care of Items 4 to 7.

For the beam-and-girder construction the omission of quartz gravel and the necessity of using expansion joint would make very nearly the same increases, bringing the cost of beam and girder construction figured on the cost of the skeleton of the building, without forms, up to the following:

Increases due to design	4.75
Quartz not allowed	4.7
Expansion joint required	3.1

These total 12.5% to which would have to be added as in the case of flat slab a considerable additional allowance because of the effect of

Items 3 to 7, Time of mixing, etc.

As pointed out before if the New York Code figured steel at 18,000 lb. the difference between our estimates under the Tentative Specifications and under the New York Code would become about 3½% greater.

If the percentages showing the increased costs under the Tentative Specifications were figured on the cost of the skeleton including forms then these percentages would become about three-fourths as large as they now show.

Section 3—Quality.

Ever since the inception of reinforced-concrete construction, the universally accepted practice has been to fix the quality of a concrete by describing the quality of the materials to be used and the proportions in which these materials were to be mixed. It is now suggested in the Tentative Specifications under discussion that a new measure be set up for the quality of concrete. It is suggested that the quality of concrete be expressed in terms of workability as determined by the slump test and of the compressive strength of test cylinders, made from the concrete as it is placed and tested wet at 28 days.

After a study of the technical literature on the subject and after trying out the suggested new method in actual practice we are forced to conclude that the quality of concrete cannot be properly expressed in terms of slump tests and of compression tests on cylinders at 28 days.

Slump tests have been in use for so short a time that there is but little published data on the subject. Our observations lead us to believe that the general sentiment of those experienced in the testing of concrete agrees with the results our members have obtained in trials in the field from which we have drawn the conclusion that slump tests cannot be depended upon to indicate water-cement ratio with accuracy. With a considerable variation in water cement ratio when using the same aggregates the slump tests will sometimes show but little variation. Occasionally with the same aggregates the wetter mix will show the smaller slump.

We do believe that the slump test is of great practical value as an approximate check on water-cement ratio, and that it can be used to advantage in helping to control water-cement ratio, but we are convinced that the slump test is not a sufficiently accurate measure so that from the results obtained by its use it will be possible to form any accurate judgment as to the strength that will be attained by cylinders made from the concrete.

This is unfortunate because owing to the variation of the water contained in the aggregate the water-cement ratio of the concrete may vary considerably even though exactly the same amount of water is put in each batch. Tests made by members of the committee showed the water content of 65 cu. yd. of mixed aggregate stored in a bin to vary from 3.4% at the top to 7.6% at the bottom on the day after a rain. Tests on 25 cu. yd. of sand in a bin showed the water content four hours after wetting down to be 5.7% at the top and 14.7% at the bottom, while gravel under the same conditions showed 2.3% at the top and 3.0% at the bottom. Variations of 2% in the water content of aggregate are very common and cannot be noticed by the eye. On reinforced-concrete building work a 2% variation in the moisture content of the aggregate would increase the water cement ration of a batch sufficiently to reduce the strength of the concrete test cylinders 500 lb. per sq. in.

We have been unable to devise any practicable way to overcome this difficulty on construction work. Wetting down the material piles does not solve the difficulty. If slump tests were accurate they could be depended upon to always indicate the change in the water cement ratio due to this variation in the moisture content of the aggregate, though unfortunately they would not furnish this information until after the concrete had been delivered from the mixer.

Fortunately the effect of a variation in the water-cement ratio is probably not nearly so large on the concrete in the structure as it is on the cylinders made for testing. The cylinders made for testing are made from the concrete as it is placed and contain all the water that was in the batch, while much water as we know drains out of concrete as it is spaded into the forms.

There are two lines of thought which will indicate the undesirability of attempting to change from the present method of specifying concrete by proportions to the method of specifying a guaranteed strength of cylinders at 28 days.

First.—The results obtained in testing cylinders at 28 days are known to be seriously influenced by seemingly trivial details in the making, storage and testing of the cylinders. Even if the rules for the making of cylinders are carefully followed the personal equation of the man making them seems to have its effect.

Mr. Greene, in a paper published in 1919 *Transactions* of the American Society for Testing Materials, has pointed out the marked effect on strength resulting from allowing the cylinders to dry out at different periods during storage. His conclusions as a result of two separate series of tests are as follows:

1—Concrete test specimens broken in 28 days which are allowed to dry out for 27 days after being kept moist for 1 day have a compressive strength as small as 40% of that of specimens kept moist for the total 28 days.

2—Concrete test specimens broken in 28 days which are allowed to dry out for 3 days after being kept moist for 25 days are often more than 40% stronger than specimens kept moist for the total 28 days.

3—The greatest variation between specimens cured properly and improperly is often over 100%.

The variations in strength noted by Mr. Greene refer only to variations due to differences in the amount of moisture in the air of the storage closet. Test cylinders are known to be very susceptible to changes in temperature. A low temperature even if considerably above freezing has a marked effect on checking the normal increase in strength.

If the results of the tests are not to be inaccurate the ends of the cylinders must be absolutely plane surfaces. It would be very difficult to get cylinders made on the average job to meet this requirement so that frequently it would be necessary to bed the ends in plaster of paris. If the testing laboratory does not allow the plaster of paris to harden sufficiently or is careless in any of the other details connected with storage and testing the test on the cylinder will naturally be misleading.

A study made by comparing the results obtained at the leading laboratories of the country, as published in the transactions of our engineering societies, will show a frequent lack of uniformity in tests of cylinders.

If concrete should be specified by strength of cylinders at 28 days it would be necessary on the average building job to make the cylinders on the job and then in many cases it would be necessary to ship them long distances by express for testing. Considering the influence on strength of the personal equation of the inexperienced men who would make the cylinders on the job and the uncertainties of temperature and dryness in storage to which the cylinders would be subjected at the job and while in transit, especially during the winter months, it is impossible to avoid the conclusion that frequently the test results obtained would be absolutely misleading. This conclusion as stated above is strengthened by an examination of the data published by expert investigators of tests made under the best conditions for securing accuracy. Tests which have come to the attention of the speaker on work with which he has been connected have convinced him of the uncertainties of the results obtained when based on the small number of cylinders that it would be practicable to make during the pouring of a floor in a building.

A hasty consideration of the problem might lead to the conclusion that the difficulty due to the uncertainties of cylinder tests could be overcome by providing that only a certain percentage of the numbers of tests made would be required to come up to the specified strength. A little thought will show that under such a specification the contractor would be required to occupy the position of a gambler as for any small number of tests the uncertainties in the testing of cylinders might lead to good results or poor results though made on concrete of equal quality.

Second.—Even if the results obtained on compression tests of cylinders at 28 days were not of uncertain reliability it would still be illogical to establish the strength of cylinders at 28 days as the sole criterion of the quality of the concrete.

It must be kept in mind that when concrete is specified by proportions the engineer writing the specification can have in his mind not only the strength of the concrete cylinders at 28 days but also the strength the concrete will have at later periods in the structure.

No comparative tests are available to the speaker's knowledge on specimens cut from buildings but those tests that have been made by the New York subway show the concrete cut from the structure to be 25% stronger than cylinders made from the concrete at the time it was placed and dried out two days before testing. If the cylinders had been tested wet by the New York subway the difference in strength would be still greater. It is entirely logical to suppose that excess of water will drain out of concrete after it has been placed in the forms and it is a fact that if the water does drain out before the concrete starts to set, then the excess water that has drained out has no injurious effect on the strength of the concrete.

A cement that gives comparatively low results at 28 days may give very satisfactory results at the later and more important periods. W. M. Kinney, in a paper on aggregates for concrete, American Concrete Institute, 1912, gives a series of experiments on five different brands of cement. The brand of cement which was weakest of the five at 28 days was the strongest of the five at six months, with a difference of about 21%.

Tests made by two members of the committee, one on nine brands of cement, and the other on seven brands of cement, all of which satisfactorily passed the tests for cement, confirm the fact that certain brands very largely used with entire success for the past twenty years will not give nearly as high results at 28 days as certain other brands of cement.

Should a specification be adopted which makes the guaranteed strength of cylinders at 28 days the sole measure of the quality of concrete, the inevitable result will be that contractors will give preference to those brands of cement that give the greatest strength at 28 days, as with such cements they can obtain a specified strength with less cement and, therefore, more cheaply. This will work a great injustice to brands of cement which are somewhat slower in picking up strength but as shown by Mr. Kinney's tests do at later periods sometimes actually exceed the quick hardening brands in strength.

Section 5—Tests of Field Specimens.

The method to be used in making, storing and testing the cylinders is given by reference to the standards of the American Society for Testing

Materials. These standards require that the test cylinder be put in the machine as soon as it is removed from the wet sand, which is not in accordance with the common engineering practice in testing concrete, from which engineers have formed their opinions of the strength of concrete. The New York subway engineers dry test pieces two days before testing. R. W. Hunt & Co. also allow test cylinders to dry out before putting them in the machine.

It has already been pointed out as stated by Mr. Greene (and confirmed by two tests on work in New York) that if the cylinders were dried three days before testing the strength of the concrete as shown by the test would often be increased more than 40%, and on an average about 30%. In laboratory investigations it may be desirable to test the cylinders as soon as removed from the damp sand in order to eliminate uncertainties as to the extent of the drying, due to variations in the air to which the cylinders would be exposed for the three day period.

While from the standpoint of the expert who is conducting elaborate comparative investigations the testing of the cylinders when wet may be desirable, it must be remembered that in this engineering specification as now written the strength of the wet cylinder becomes the basis of determining the unit stresses to be allowed in design, and therefore unless the cost of work is to be very materially increased it will be necessary to provide that the strength of the concrete is to be judged by tests on cylinders that have been dried three days before testing.

The measure for the strength of concrete has been made more and more restrictive. Twenty years ago ideas as to the strength of concrete were based on tests made on cubes. The next step was to change to tests on cylinders which in engineering practice were tested dry and gave strengths about 75% as large as had been obtained on the cube. This change in the form of test piece was not made because of any difficulty experienced with buildings in which concrete had been passed as 2000 lb. concrete by the tests on cubes. The change to the cylindrical form of test was made because in laboratory investigations it was found to give more consistent results than could be obtained with cubes. It is now proposed to specify that strength shall be measured by tests on wet cylinders which will mean a further reduction in indicated strength of some 25%. The suggestion to make this change is not because of any experience with concrete buildings which would make it necessary to require a richer mix of concrete, but simply to secure a more uniform method of testing.

When the amount of building that has been done with entire success with 1 : 2 : 4 concrete figured at 2000 lb. is considered, and it is further noted that through the more careful control of water ratio concrete placed in the future should be even better concrete than the concrete placed in the past, it is difficult to understand why a specification should be proposed which would make it necessary to use a mix of about 1 : 2 : 3 to obtain 2000 lb. concrete, unless it is the intention to increase the allowable working stresses in a proportionate amount. As will be shown later, comparative estimates show that the allowable working stresses and assumptions made in design have not been changed to compensate for the proposed new measure adopted for the strength of concrete and, therefore, as the specification stands its adoption would lead to an unwarranted increase in the cost of buildings.

Section 27—Method of Measuring.

We recommend that the specification provide for measurement by volume without reference to measurement by weight. Apparatus practicable for measurement by weight is not readily available on the usual run of work. Departures from accepted practice should not be included as alternates in a standard specification intended for general use. Where alternates are allowed there is sometimes a tendency on the part of those not having a thorough knowledge of the industry to use the alternate which seems to provide the most scientific and exact determination without appreciating the difficulty or extra cost involved.

Section 28—Proportions.

Alternate 1 attempts to specify both the proportions in which the concrete shall be mixed and also the strength the concrete is to have. It is impossible to specify both of these conditions without first so modifying the working of this alternate as to make it equivalent to the provisions now contained in Alternate 2.

Alternate 2 requires the contractor to fix the proportions and to guarantee that the concrete will have a certain strength when tested in wet cylinders at 28 days. It must be kept constantly in mind that while this is only one alternative method of specifying proportions, still if it is left in as an alternate probably many architects and engineers will select Alternate 2 as the basis of their specifications without appreciating the difficulties involved by such a selection. Among these difficulties are the following:

It will be impossible for a contractor to accurately estimate work in a new location because he will not know in what proportions his concrete will have to be mixed.

When the work is in progress, the tests of the concrete put into a floor may fail to come up to the specified standard. In this case, the building department may refuse to post the floor for the live-loads for which it was designed. The contractor will claim that the concrete he put into the building was perfectly good and that the fault lies with the making and testing of the cylinders by the engineers. Those who have studied the results obtained in tests on cylinders, know the uncertainties of such tests when made in limited number.

It may be taken as a general principle that any proper specification will be so drawn as to permit the engineer to inspect the structural parts of a building as they are put in place so that he may know at the time the work is done that it will be acceptable. The interests of the owner, of the contractor and of the engineer all require that this condition be insisted on. Under Alternate 2 the engineer would not know whether a floor was acceptable until a month after it had been put in place, or until three or four more stories had been superimposed on the floor in question. Under such a condition the engineer who was compelled to inform an owner that the floor was rejected and then explain that his specification did not enable him to know whether work was good or bad until a month after it was put in place, would occupy an intolerable position. A specification which does not permit the contractor to inspect his material in advance of its use and then know he is complying with the specification when he places that material in the manner required by the specification, also creates an intolerable condition for the contractor, as it subjects him to the danger of unjust exactions.

Instead of operating to the advantage of the honest contractor and to the disadvantage of the dishonest contractor, Alternate 2 would seriously hamper the honest and conservative contractor while assisting the unscrupulous or careless contractor in obtaining work in competition.

Everything that it is reasonable to wish to obtain by the use of Alternate 2 can be secured by the use of Alternate 3 when modified to provide that the engineer shall change the proportions when he considers it desirable to do so, with adjustment of the contract price to be made if the cost of the work is increased or decreased by such change. The reference to Table IV should be eliminated from a specification.

Section 31—Time of Mixing.

Recent tests seem to show that the gain in strength due to mixing one and one-half minutes, instead of three-quarters of one minute is probably about 5%. Such a small gain in strength does not warrant continuing the mixing beyond the three-quarter minute period in cases where the mixing water is introduced promptly. While a fraction of a minute seems a small matter, a slight change in the time of mixing will frequently have an important effect on the amount of plant required. The one yard mixers now used by contractors on large buildings are about as large as can be placed in basements and, therefore, if the time of mixing is extended it will frequently mean placing less concrete per day in cases where monolithic finish is used, or installing a second mixer.

Section 40—Protection.

This section should be changed to provide that the concrete shall be kept thoroughly wetted when practicable. It would be very difficult to keep certain portions of the concrete in a building wet. As yet we have no experiments on the strength of concrete cut from buildings to show that the concrete in the columns, etc., does not gain strength in a satisfactory manner under the conditions that exist in a building during construction. Nothing should be done in this specification to increase the cost of a building without first having proof that the increased cost is made necessary by an actual requirement to obtain good work.

Section 41—Cold Weather.

We recommend that this section which broadly prevents the addition of salt or other chemicals to prevent freezing, be modified so as not to shut the door on the developments which indicate that for certain work where electrolytic action is not to be feared such chemicals as properly prepared calcium chloride prevent freezing cheaply and without damage to the work.

Section 67—Fire Protection.

We believe this section as written will prevent the further use of quartz gravel concrete, because of increased cost due to the requirement that where quartz is used the fireproofing must be increased one inch and a metal mesh must be placed one inch from the outside surface. As drawn this section seems to apply to all parts of a building, without recognizing the fact that columns may require a different treatment from that given slabs and walls. We doubt if this restrictive condition is necessary except for spiral columns in buildings where the fire hazard is unusually severe, and in that case it is possible that study might show a more economical method of overcoming the difficulty. The increased cost occasioned by this section, applying as it will to all buildings erected in cities where quartz gravel is the cheapest available aggregate, must be compared with the greater fire loss that will occur in the comparatively small number of such buildings that will be subjected to fire.

It is also necessary to keep in mind that the man who builds will use what he thinks is the cheapest construction suitable for his purpose, and therefore if the use of concrete is made too expensive, many owners will turn to the use of timber construction or some form of semi-fireproof construction. It may be, therefore, that the practical prohibition of quartz gravel concrete will lead to an increase in the use of less desirable forms of construction.

Before penalizing quartz concrete so severely, a most careful study should be made of the results obtained with quartz concrete for columns and for flat surfaces as compared with other forms of fireproofing that are now in use.

Section 75—Expansion Joints in Long Buildings.

A properly constructed expansion joint is expensive to construct and is also undesirable from an operating standpoint. The speaker believes that buildings 300 ft. long or longer can be built without expansion joints and without danger of injury unless the section of the building at some point near its middle is very much reduced by openings. The spacing of expansion joints depends so much upon the details of design that it seems undesirable to put an arbitrary limit in a standard specification.

Section 79—Integral Compounds.

We recommend that this section be modified to permit the use of integral waterproofing compounds under proper restrictions.

Section 80—Membrane Waterproofing.

We believe that hydrolithic waterproofing can frequently be used to excellent advantage and therefore we recommend that a section be introduced allowing the use of hydrolithic waterproofing under proper restrictions.

Section 94—One-Course Work.

Many excellent floors have been constructed by sprinkling a mixture of dry sand and cement on the surface of the concrete. Some of the largest manufacturing companies of the country now specify this method for buildings erected for their own use. We feel that this method should not be prohibited, as might be inferred from a reading of Section 93.

Section 95—Two-Course Work.

We feel that to require the placing of a monolithic wearing course within one-half hour after the placing of the base course is unreasonable. We recommend that this section be reworded so that the time that may elapse is made dependent on the temperature, humidity, setting time of the cement, etc. This might be done by specifying the condition of hardness the base course can have at the time of spreading the wearing surface.

Mr. Humphrey. RICHARD L. HUMPHREY.—Mr. Dixon has given us a very detailed criticism of the recommendations of the Joint Committee. I would like to ask him what he would propose to substitute in lieu of what they recommended. Is it his idea that the proportions of concrete shall be determined in advance and then the concrete put in with varying degrees of water, varying from stiff to a very sloppy consistency, or what would he provide in the specifications?

Mr. Dixon. DEF. H. DIXON.—We feel that the specification of concrete should be by proportions. The engineer should specify that the concrete for the building is to be 1 : 2 : 4 or 1 : 2½ : 5 or 1 : 1½ : 3, or anything that he feels is proper.

A Member. A MEMBER.—Regardless of consistency?

Mr. Dixon. MR. DIXON.—No. It should be provided that during the construction of the work slump tests should be taken. If the slump tests show too much slump, that is brought home to the contractor right away, and he is told that he must get a slump that is not so great. I am sorry that I have to take so much time in what is criticism without having time to dwell on the advantages of what has been found out, if it is only used in an educational way to educate engineers and contractors, instead of being written into a specification which will penalize contractors so heavily.

Mr. Humphrey. MR. HUMPHREY.—As I understand, you would put the responsibility for the quality of concrete on the engineer who represents the owner?

Mr. Dixon. MR. DIXON.—That is correct. He is supposed to specify the way that building is to be built and to inspect it and see that it is done right.

Mr. Humphrey. MR. HUMPHREY.—And yet the quality of that concrete in a very great measure depends entirely on the manipulation, which is wholly under the control of the contractor.

Mr. Dixon. MR. DIXON.—That is all right, but there are ways of judging the strength of that concrete while it is going in as accurately as it can be judged. The engineer has any way that is open to the contractor to judge the strength of that concrete. It is open to the engineer or his inspector just as he inspects anything else. He should have the results of slump tests, and if they are not right, the engineer can stop the work just the same as he can stop the work if the contractor does not do anything else that is right.

Mr. Humphrey. MR. HUMPHREY.—Has your New York Committee framed any paragraph as a substitute for Section 28? As I understand your criticism, none of the alternates are acceptable.

Mr. Dixon. MR. DIXON.—We said that alternate 3 was all right, modified so that the engineer specifies the proportions and is at liberty to change those proportions if the modification requires it.

W. A. Slater.—There are three proposed alternate methods of specifying concrete in the tentative specifications. One of them proposes specification of strength alone; one of them after having predetermined the strength of the concrete specifies the proportions to be used, based upon the proportions and materials used in those tests; the third is the method which is now generally in use, that of merely specifying proportions and applies only to cases where no preliminary tests have been made. As I have understood the discussion given here, the desire is to throw out the first two provisions and practically leave the specifications for, and proportioning of, concrete just as they are generally made and carried out at the present time. Mr. Slater.

There can be no question that some improvement is needed in our method of specifying and mixing concrete, and the first two alternatives proposed are for the purpose of securing that improvement. The argument against those two specifications has been mainly that concrete varies in strength, that we cannot get the strength that we specify, and that, therefore, merely to specify proportions should be specified and the fact covered up that the specified strength is not obtained.

I do not believe that those who have discussed it intend to put it in that light; I do not believe that it is their intention to slight the work, and I want to ask if this would be acceptable as a proposed addition to the specifications, in order to make either of the first two alternatives acceptable; suppose that as the final settlement as to whether the job is acceptable, a test of the structure could be devised that would settle the question of whether it should be accepted or not. Suppose that the alternative under which the strength of the concrete is specified is used on the work and when the control specimens of concrete are tested the strength falls down, as a final criterion as to whether the work is acceptable, would not the actual test of the structure be acceptable? It would not be desired to test every building, but in case the concrete did not develop the strength intended and it became necessary then to show that the work was satisfactory, to show it by making a test on a represented portion. If the structure is a floor it is not difficult to see how it could be tested; if it was a retaining wall, it would be rather difficult to test it; but if such a test could be devised, would that permit the use of the alternative aimed to improve our methods of working and at the same time take care of the difficulties inherent in them?

Mr. Dixon.—I do not see the necessity of making a test on every building built; it does not seem necessary. Mr. Dixon.

Mr. Slater.—The purpose would not be to make a test on every building that is built, but in case the contractor had not produced a concrete of the strength guaranteed and it became necessary then to show that the work was satisfactory, to show it by making this test. Mr. Slater.

Mr. Dixon.—That would be unfair to the contractor in this way, that these tests are so misleading that very many times he would be required to make this test when it would not be necessary at all. I have so little Mr. Dixon.

Mr. Dixon. confidence in any small number of cylinder tests, that I feel that many times there would be an unjust suspicion against the contractor, that his work was not right according to the specification. The engineer would say "These tests have not come up to what is required; we have got to make a test now and see if the building is all right." The owner would feel that the contractor had slighted his work, when there would not be any justification for that feeling at all.

Mr. Humphrey. MR. HUMPHREY.—If I gather your thought correctly, you have a belief that wet cylinders test a great deal lower than the same cylinders after they dry out. You also believe that if the concrete that goes into the structure is to be tested by cutting out a specimen of the concrete that goes into the structure, you would get a higher test than from the cylinder; you feel that if the proportions are specified in advance as provided in alternate 3, that would be sufficient if tests were made to get the necessary quality of concrete that should go into the structure.

Mr. Dixon. MR. DIXON.—Building work is not at all as it is in large dams and work of that kind. In building work we have to start with about one inch maximum size aggregate, and the chances you have for grading from one inch down are very much less than they are in the case where you are building a large dam and start with 2½ or 3-in. aggregate. With 3-in. aggregate you can effect considerable economy by grading, but when you start with 1-in. maximum aggregate, in order to work grading for economy, you have to go way down to the very fine stuff, stuff that you cannot screen wet to get the full advantage of grading. You can do it in the laboratory, but you cannot do it in a sand and gravel plant or on a job, and the sand and gravel plants also are not equipped to do it. So you do not have a chance on a reinforced-concrete building, to make the economies through the grading. I have much more confidence in fixing proportions to use in building on proportions than on any tests you could make.

Mr. Humphrey. MR. HUMPHREY.—If I understand your proposal, if you write in alternate 3 as you propose, you have not made any step forward in improving the quality of concrete, you have simply put in specification form current practice which is not giving us the quality of concrete we should have.

Mr. Dixon. MR. DIXON.—There is nothing that prohibits the engineer from specifying the making of tests. It is a matter up to the engineer, not the contractor, so it should not go in the specification. It is expected that the engineer on the job will have cylinder tests made and if they do not come out right he will see that the work is corrected through his inspection.

Mr. Hatt. W. K. HATT.—It seems to me there are two questions here. The first is as to the state of the art of testing. I think we might concur as to the technique of cylinder testing in the field. Apparently a good many think that it is not precise enough to be the basis of a contract. The other phase of the question is this, the contractor is to supply, as I understand it, materials up to specification and workmanship. I do not think it is the business of the engineer to specify the materials as much as it is to design the building. I do not think it ought to be left to the contractor to design the building as a matter of theory. As to workmanship, is there anything that the contractor can do to judge of the quality of the concrete as it leaves the mixer, which the engineer cannot do as well? He is trained for that purpose; that is his special function. I do not see what advantage there is in transferring the function of the engineer to the contractor. It is true that there are contractors who are engineers, and they have as great skill and as great experience as engineers, but there are not many of them, and while I appreciate the need for the improvement of concrete as much as any one, I do not know that we are going to come at it by establishing a utopian theory or a doctrine which cannot be applied as the world goes along. **Mr. Hatt.**

SANFORD E. THOMPSON.—Most concrete which is laid in reinforced-concrete buildings and is supposed to be 2,000 lb. concrete, tests around 1,200 to 1,400 lb. per sq. in. at 28 days. This is a strong statement, but it is made as a result not of theory but of many practical tests on cylinders that have come from jobs in all parts of the country in commercial practice. This low strength is largely due, as is generally recognized, to the consistency of the concrete. It is what we are up against, and you will note the wide margin between say 1,400 lb. and 2,000 lb. The question then is how we are going to better the conditions. With reference to the taking of cylinders, that is, the making of cylinders on the job, I do not agree with Mr. Dixon as to the impossibility of getting good specimens. On some very large structures that we have been associated with, for example, we found it possible to get specimens which tested remarkably uniform. If 2,000 lb. concrete was specified, we found it possible to get cylinders which, except for cold weather, averaged almost exactly 2,000 lb. per sq. in. For other proportions we got average results which corresponded very closely with the theoretical ratios. This was obtained by following carefully prepared specifications for taking the field specimens, specifications that were afterward used as a basis for the Standard Specifications for Field Specimens of the American Society for Testing Materials. **Mr. Thompson.**

The cylinders were stored on the job until hard, then taken to the laboratory and stored under a moist cloth until just as they were tested. Our experience has been that field concrete, that is, specimens taken from the concrete poured into the forms, averages stronger than similar specimens of similar consistency made in the laboratory. This makes this criticism of field concrete all the greater.

L. C. WASON.—I would like to ask Mr. Dixon if he does not think it is fair and reasonable to insist on the portland cement companies selling cement on a compression test instead of on the usual tension briquette as the present specifications call for? Secondly, what do you think the reaction of the cement companies would be on such a proposition? **Mr. Wason.**

Mr. Dixon.

MR. DIXON.—The committee considered that, and we quoted in our report a recommendation. Of course, if such a specification is adopted, contractors would expect that cement would be accepted on a compression test, and our compression tests are made on concrete mixtures, but that really does not answer the question at all, because those tests would merely give the data which you could get yourself, and you would still find that certain cements would give you greater strength in 28 days than other cements, even though they might not be any better at three months and six months. Therefore, as you were only interested in the strength of the cylinders at 28 days under this specification, you would not have to care a particle about the quality of that concrete in the building in three months; the only thing you are interested in is "What do cylinders test in 28 days?" So you would still buy those cements that would give you the high strength in 28 days.

Mr. MacMillan.

F. R. MACMILLAN.—There is one angle to this matter that I think ought to be emphasized. It was in the minds of the committee that in offering these alternate clauses 28, they were making available to the contractor an opportunity for taking responsibility. It was not their idea to pass it from the engineer to the contractor, but to offer him an opportunity for increased economies by careful selection of his material and methods, thereby giving him greater liberty to serve his clients at a better profit to himself. Apparently that phase has not been appreciated by the contractors' speakers, but I would like to have that clearly understood. It was in the mind of the committee to offer an opportunity for effective economy, and a great many of us still believe that if the contractors cared to avail themselves of that opportunity, they would demonstrate the economy of it.

I note, too, that the New York committee selects as a standard the code of the city of New York. Why stop there? Why not pick out the code of another city? Why do we emphasize the code of New York as a standard after it has been shown that some of the other city codes are a good deal more expensive as compared with the report of the Joint Committee? It has been assumed that the New York code was the one perfect code and anything above that was an increase and a burden on building construction. Why not take the Minneapolis code or some other code under which we can allow three times the load we allow under the New York code?

III. PHILADELPHIA, PRESENTED BY F. V. WARREN

Section 28.

This section provides methods for determining the proportions in which the cement and aggregates shall be mixed.

The first division provides both the proportions in which the concrete is to be mixed and the strength to be obtained as determined by test cylinders taken from the work and tested at 28 days.

The second division leaves the proportioning to the contractor and requires him to produce concrete of a given strength as determined above.

Only the second of these sections would be possible of operation, as it would be manifestly unfair to require the production of concrete of a certain quality without the privilege of varying the proportions from a given arbitrary formula. We believe, however, that any attempt to impose even this condition on the contractor would lead in many cases to controversy and litigation.

We are aware that under present conditions wide variations are shown in the strength of test specimens taken from apparently identical mixtures of concrete. We believe however (and feel sure that we will be supported in our contention) that this irregularity is due to the varying properties of the fine and coarse aggregates and to the water ratio. When the grading is determined and the consistency controlled, uniformity is obtained.

We earnestly recommend the installation and use of the proper water measuring devices, and the frequent and regular use of the slump test or any other workable method of controlling the water ratio, but we believe the scientific determination of concrete mixes to secure a given strength is the function of the engineer rather than of the contractor.

The engineer should know the available sources of supply for any structure he is designing. He would make one investigation. Whereas, if it is left to the contractor as many investigations are required as there are contractors bidding.

We believe the results sought by Section 28 can best be obtained by more detailed and rigid requirements of field operation.

We believe further that confusion would be likely to result from the judging of concrete entirely on the compressive strength of test cylinders at 28 days, due to the varying conditions under which tests may be made and to the fact that many cements do not give high compressive tests at that period but do over a long time.

We strongly recommend that the standard specifications contain only the method of proportioning by the volume in which the cement, fine aggregate and coarse aggregate are to be mixed. We believe provision should be made for the engineer to modify the proportions when necessary to do so with an adjustment of the contract price, consistent with the change.

Section 31.

We recommend that Section 31 be revised to require the time that a batch should remain in the mixer to be ¾ minute after all the materials are in, and that locking devices be required to make this time positive.

Section 67.

We strongly recommend the elimination of that portion of Section 67 which provides for 1 in. additional fireproofing and the placing of a metal mesh 1 in. from the outside surface in all concrete where aggregate is used having an expansion materially greater than that of limestone or trap rock. This provision would eliminate the use of quartz gravel. In some localities this would so increase the cost of reinforced concrete as to prevent its use.

We doubt if this restriction is necessary except in rare cases where the fire hazard is unusually severe or a building is to be used to house highly inflammable material. These cases should require special study and treatment, and it should not be necessary to impose an increased cost on all buildings to provide for an increased risk in the comparatively few.

We recommend that in all concrete structures there should be 1½ in. of concrete protection for all steel in portions exposed to the weather.

DISCUSSION OF PHILADELPHIA REPORT.

ERNEST ASHTON.—I note that most of those who have discussed this matter this morning view with considerable alarm and apprehension the great variation existing in concrete test specimens, notwithstanding the fact that the control of the manufacture of those specimens is always in hand, always definite and always positive so that the variables can easily be eliminated one at a time. They view with alarm the great variation existing in the results and the concrete. I wonder whether they view with the same alarm and apprehension the concrete that they now have in structures where there is no such thing as control, or very little? **Mr. Ashton.**

DUFF A. ABRAMS.—To pursue the point raised by Mr. Ashton, we have found in our investigational work that concrete tests, if made under uniform conditions, are as reliable as tests made on any other material with which I am familiar, with the possible exception of mild steel. If we make five or ten tests alike and get a variation of more than five or six per cent, we begin to look for the trouble, so that if concrete tests do vary, it means that there is a variation in the concrete. If Mr. Dixon finds that the tests with a higher water ratio give a higher strength, I do not believe it has any relation to the validity of the water-ratio relation, but simply means that the concrete itself varies. Concrete is a very dependable material, if it is tested consistently. **Mr. Abrams.**

H. C. TURNER.—May I ask Mr. Abrams in connection with these tests which can be made in the laboratory with such a small variable, is that comparison always made with one brand of cement, or do you find the variation mentioned by Mr. Dixon between the strength obtained in concrete mixtures where there is a variation in the cement itself? **Mr. Turner.**

MR. ABRAMS.—I was speaking of the variation in tests made with the same materials, same mixtures and under the same conditions. We have, of course, tested a great many different brands of cement and there is a great deal of information available on that subject that may not have come to the attention of Mr. Dixon's committee. If he will look up the report of the cement committee published three or four years ago, he will find a large mass of information on that very subject. Our laboratory, I think, made tests on twenty-three brands of cement, all in the same condition. They included the ordinary cement test as well as tests of cement mortars in compression, and compression tests of concrete of different mixtures at a number of different ages. Those tests were carried out under the supervision of Am. Soc. T. M. Committee C-1, on Cement, at the time the last revision in the specification for cement was made. **Mr. Abrams.**

S. E. THOMPSON.—I would like to ask Prof. Abrams in a general way how the relation of the compression test of concrete with different cements ran? **Mr. Thompson.**

MR. ABRAMS.—I do not recall just what that showed; it has been **Mr. Abrams.**

Mr. Abrams. some time since I have seen a report; there was some variation no doubt, but I think it would be measured by probably a matter of 25 or 30 per cent as between the lowest and the highest.

Mr. Ashton. ERNEST ASHTON.—I think the total range on the series you referred to, the original series of the twenty-three brands, the total range was 65%. Of course, there may be an actual reversal of these conditions by taking different samples at different periods of time. These samples were purchased on the open market and sent into the laboratory for test, the primary study being the relation between the tension and the compression test. They were only individual samples purchased on the open market.

Mr. Hatt. W. K. HATT.—It is common practice in laboratory investigations to use a mixture of four or five brands of cement to eliminate that variable.

The question has come up as to the significance of the variation in tests of field cylinders, and the implication of the discussion has been that there is the same amount of variation in the concrete in the structure. I would like to know just what the facts are. We have a great many cores taken from concrete roads. It is a practice that is increasing, and the indications are, as I think, that the variations in cores from the road are quite as great as the variations in the cylinders cast at the same time. We hear a lot of talk about concrete having what they call mass strength and a sort of evening up of conditions when it is poured in large masses. I think before we can make up our minds on this particular matter, we ought to study and increase the number of tests of cores taken from construction. I do not think that has been done in the case of buildings to a great extent, but we can answer those questions by taking cores out of construction.

Mr. Nichols. C. E. NICHOLS.—It is obvious that the contractors' relations must be protected, his pocketbook cannot be sacrificed entirely. I want to add to the discussion this much as coming from a contractor with a considerable engineering viewpoint: We contractors must admit that we have been getting very uncertain results from concrete; we admit that the cement used varies considerably in concrete work; I do not see that that is any argument whatever for not taking what steps we can, that research indicates as advisable, to correct the faults relating to the other materials in the concrete. I think we should go after the cement question also, but I do not see that that is any argument for setting aside the research which has been conducted and which indicates what can be done with proportioning with sand, gravel and water. Any of the contractors who have closely followed this test will admit that the results Prof. Abrams read a moment ago are by no means unusual. The average of what we have thought was 2,000 lb. concrete probably will not run as high as 1,600 lb.

I think another thing that can be developed along this line and which must be developed to follow out the recommendations of the Joint Committee, is a considerably higher degree of accuracy in the technique of field testing and field specimen making. I think a great deal of the

Mr. Nichols.

uncertain results that we have had from tests made by contractors on the job is due to the lack of experience along this line. I know that has been indicated by two or three jobs in which we have attempted to follow somewhat along the lines recommended by the committee. The results were anything but what might be expected and were undoubtedly due to the inexperience of the men making the tests. That is where we must devote more of our energy.

One thing that Mr. Wason brought up in the beginning I want to touch on: He advocates a specification which specifies results rather than detailed methods—that is, in discussing specifications other than Section 28. I think that Section 28 as it stands now, or the alternates of Section 28, and the criticisms that have been brought up against it are wonderfully good arguments against his sort of specification. You can specify the results you want to get and leave it to the contractor to get them, and you will be just in the position of the engineer who specifies "make 2,000-lb. concrete," and when you do not get it, you simply haven't it. I think it is absolutely essential that a good specification turned out by an engineer should specify not only the results he wants to obtain, but the detailed methods of getting them, and if the contractor wants to use some other methods, it is up to him to get the consent of the engineer who wrote the specification and it is up to him to assume the responsibility for getting the results if he changes the methods prescribed.

IV. Cincinnati, Presented by F. N. Loring.

The deliberations of the Cincinnati committee resolved themselves into a conference between myself and Mr. Henley, a consulting engineer in Cincinnati. We have a committee that has been working for some time on the matter of revising the concrete sections in the building code, and when this matter came up it was suggested that their interest in the subject would make their opinion of the proposed regulations of interest. It has been impossible to get all of the members together, so I am reporting what Mr. Henley and myself have agreed upon as we went over the matter in a rather short time.

We put up to ourselves specifically the time of mixing as specified in Section 31, and our recommendation was three-quarters of a minute as a minimum. In regard to Section 28, I think it has not been brought out that that is intimately connected with Section 4, which specifies the strength of the concrete. We feel that Section 4 should state that the design is based upon assumed strengths, and that the third alternate in Section 28 should be substantially adopted and leave it up to the engineer to make field tests and to change the mix specification if he feels that it is necessary to do so in order to get the assumed strength of concrete which he used in his design.

We have in Cincinnati really only a few aggregates, and we feel that there is no reason in our city why an engineer should not be thoroughly familiar with the results he can expect from certain combinations of those aggregates. The principal aggregates are river gravel, bank gravel, river sand and bank sand, and while we think that further tests should be made in the light of our recent information on the behavior of concrete with different degrees of water ratios, that that information ought to be forthcoming and standardized so that engineers in our locality can specify a mix of concrete with some confidence in the results that they will get.

DISCUSSION BY A. C. I. COMMITTEE CHAIRMEN.

REPORT ON COMMITTEE C-6, ON FIELD METHODS,

PRESENTED BY N. M. LONEY.

As soon as the Joint Committee's report appeared with this specification it immediately quickened the interest which I had had for a long time in trying to overcome the waste which undoubtedly exists now due to irregularity of our results. As I see the whole matter, the object aimed at is very laudable and some day we will probably accomplish it, but at present the state of the art is such that we are simply asking the contractor to perform an impossible task, one which he would be foolish and suicidal to enter into in a contractual relationship.

Before I get to Committee C-6, I want to say something about Section 28. I think that everything aimed at in that section can be accomplished by the engineer. Fundamentally it is the engineer's function to design the mix of the concrete as well as the size of the beam. The contractor should be prepared to give that engineer all the tools he considers necessary to arrive at his conclusions as to the design of the mix. As to a definite recommendation on Section 28, I should say that the third clause should have some additional means of measuring the consistency specified, either the slump test or possibly some better rule can be found; then let the engineer specify what might be termed a base mix, including a very close definition of consistency; then, using the tools which the contractor should be required by the specification to furnish, the engineer should make as many tests as he deems to be necessary, and should then, from time to time, if you please, direct the contractor as to the variation in the design of the mix.

Committee C-6 has, as its first recommendation, that all persons concerned in the production of concrete continue the work already started and which, I believe, has progressed a great deal in the last couple of years, namely, the education of our rank and file to the necessity of regulating consistency. We have in the industry men who have been putting in concrete for ten to twenty-five years, who have actually produced, on the whole, creditable buildings. These men have, until a short time ago, been instructed to pour, that is the word, "pour," instead of place concrete of sloppy consistency. This education, I think, is proceeding now and should be continued. That is the first recommendation of Committee C-6. Every one concerned, all of us here no doubt can personally influence that situation if they do as I do as I go about and preach this doctrine.

In the next place, the committee believes that further progress is going to be largely dependent upon better tools which will remove the human element as far as possible. If, for instance, it were possible to have a mixer which would automatically reject the surplus water, then we could write such a specification as has been proposed and have some hope of carrying it out. That, of course, is probably a mere utopian dream, but undoubtedly a lot of improvements can be made and there is no doubt a closer proportioning of aggregates can be made. That is merely a matter of developing some equipment which is practically in a commercial state now. There is one notable job which has been more or less written about and will be more written about, namely, the Queenston Chippawa power project at Niagara Falls, where a considerable saving was made in cement by the adoption of a simple rule which, as far as I understand it, boils down to this: Every time you put six gallons of water into your mixer, you put in a bag of cement and increase the cement as you find it necessary to increase the water to get the proper consistency to enable the concrete to be placed.

As to the steps which we hope will develop more accurate proportioning of materials, it appears in the present state of the art that proportioning by volume is all we can hope for. It may be that, at least on operations of some magnitude, proportioning by weight, when proper tools are provided, will be possible and desirable. The development of an apparatus for compensating for the volume of entrained moisture in the aggregate is possibly another utopian thing, but the suggestion has been made, based on Prof. Hatt's test, that there is possibly a relationship between consistency and the power required at a certain place in the mix. The committee proposes to institute some investigations along that line.

On operations of sufficient magnitude to justify it, the greater use of what might be called field laboratory methods will be also advocated. All of these things are desirable and as matters stand now, a lot of us are doing something along this line and spending some of our company's money for what we think is the ultimate good of the industry, at the present expense of the same. As a contractor, speaking for one concern, I think that instead of all of us spending considerable money along parallel lines, it may be possible to fix up an arrangement whereby somebody can be paid to devote attention to this.

DISCUSSION OF COMMITTEE C-6 REPORT.

Mr. Chace.

W. G. CHACE.—These matters of mixing and design of mix have been uppermost in my mind for the last ten years. In 1913 I was placed in charge of the work of building a 96-mile aqueduct for the Water District at Winnipeg, where we had to put in about 350,000 yd. of concrete, and the design of the mixture was the first subject of our attention. The primary object we had there, however, was watertightness, but whether it be watertightness or strength that is under consideration here, the principles applied might be interesting. We spent six months on laboratory tests studying economical methods of insuring watertightness and coincidentally being sure that in getting watertightness we did not sacrifice strength beyond the limits necessary for our structures.

As a result of our tests we decided upon a mixture of aggregate containing plenty of very fine sandy particles. We were so fortunate as to find a natural deposit of proper material and, owing to certain physical conditions, to induce the owner not only to specify the mix, which was required, but also to provide the material ready for the mixer. Wherever the work was made, whether at mile one or at mile twenty-one or ninety-one, the same materials in the same proportion were delivered to the contractor ready mixed for his cement and water. On that job it was possible to avoid irregularity and we got results that were uniform from day to day and from mile to mile. That was a peculiar situation and one which would not necessarily obtain on other jobs, but the regulation of the mixture is not at all beyond the control of the engineer himself.

The question of mixing and time of mixing was one that came up before us, and at first we specified two minutes. It was not long after we got the work going before we cut that time down considerably and allowed the inspectors to determine when the mixture was sufficiently prepared. The company with whom I am now associated was one of the contractors on that job, and their governing rule is this, "to mix the concrete *until it is mixed.*" That is about as logical a rule as you can find. Any effort to govern the time of mixing by the clock is sure to fail, unless it be made an extravagant and impracticable time.

A very important feature about the whole matter is the order of placing the materials in the mixer, and that is a thing that has not been given any attention in the discussion this morning. Our company's practice is to make a grout; first the water is introduced and the cement, then the other materials, the intention being to make sure that every particle of sand and stone has the opportunity to become coated with grout as well as mixed with the grout, and we have governed ourselves in three or four ways: we use an open mouthed mixer in which the mix can be constantly inspected; it is not done blind; we dump from that mixer into on open hopper where an attendant who handles the gate of the hopper has an opportunity for touching up any little sand or anything that may

Mr. Chace. come through the mixer not properly mixed with grout; and finally we use a bucket which actually gives the material another mix as it exudes from the bucket.

Prof. Talbot. PROF. A. N. TALBOT.—At the meeting of the American Society of Civil Engineers in New York in December when this matter was discussed, I gave my views in regard to these three methods of proportioning. I do not want to repeat what I said then. I will say, however, that it seems to me that no one of these three should be accepted. They do not lead to proper and satisfactory contractural relations, they are not practicable for carrying out, as I view the way they would be used. I wish to say also that I am not ready to accept methods of specifying materials or of designing concrete based merely on sieve-analysis and water-cement ratio, with the knowledge of these matters that has been given out so far. The state of the art it seems to me has not reached the place where we can accept such an advance as is implied in these specifications.

I do want, however, to say something more about the third one of the provisions, that which specifies that the proportions shall be those given in Table IV, since both Mr. Wason and Mr. Dixon referred to this as being an acceptable method. I felt when I saw the table that it was an uneven table. As I studied it and made calculations on certain parts of it, it seemed to me that it would result unevenly in strength and unevenly in workability. I tried that out. Selections were made, not in the extreme portions of the table but in the middle portion of it, parts that I think come under material quite commonly used. Two sands were used, one running from zero to ¼ in., an sand that we call in our laboratory Attica sand from the Wabash River, a very good grade of sand; I think anyone here would consider it a high-grade sand; another sand coming from Covington, Indiana, a finer sand running from zero to No. 14 sieve. The coarse aggregate was gravel, well graded, and there were two sizes used, one, ⅜ in. to 1 in., and the other from ⅜ in. to 2 in. Three specimens of each mixture were made, and at different times. Three consistencies of the table were used, in the first one the slump ran from ½ in. to 1 in., in the second from 6 to 7 in., and the third from 8 to 10 in.

These tests were made by approved methods, by experienced workmen. and were carefully made. The individual specimens with the exception of two sets, I think, varied but little from the average; the conditions were uniform and the material itself was quite uniform. Twelve of the mixtures were chosen from the proportions of Table IV for 2000 lb. concrete, and twelve of them for 3000 lb. concrete. With each of the two kinds of sand and each of the two kinds of gravel, the mixture for each of the three consistencies was taken. As the specimens were made up a slump test was made for every single specimen, so there was no question of filling that portion of the requirement of the table.

Now the strength of these mixtures at 28 days, as I think might have been judged, varied considerably. Let me read the values for the 2000-lb.

concrete, giving only the nearest hundred pounds, each value being the average of three specimens: 1800, 2400, 1600, 1500, 1800, 1500, 1800, 2400, 2900, 1300, 1900, 1700, and for the 3000-lb. concrete, 3400, 4000, 3900, 2600, 2600, 3200, 3300, 4000, 3100, 2800, 3200, and 1900. The variation in these cases, as you see, was such that the highest was double the lowest. Now this material was not unusual material; it was what I should say would be considered a good grade of material and of course the material had been screened so that it was far more uniform than material on a construction job would be. It seems to me then that it would be unfortunate if this report should go out with Table IV put down as a requirement for strength. I think that this objection should be carefully considered. Mr. Talbot.

W. A. SLATER.—I wonder if Prof. Talbot would be willing to suggest what method should be used, since none of the three methods proposed here seem to be satisfactory? Mr. Slater.

PROF. TALBOT.—I think I should put that the other way, that it is up to the committee to do that. I has proposed a method which, according to the views expressed here this morning, seems to be revolutionary, and objections are made. We have not had an opportunity yet to hear from the committee whether these are really objections or if they are objections, how the method may be modified. The committee should have the opportunity to give us their views now. Personally I hope we shall advance in this matter further in the near future. I should rather stand on the old 1:2:4 than do the things that have been proposed. I think too that some method may be devised by which it may be specified in advance that so much cement per cubic yard shall be used, and that the mixture shall be by certain proportions of the materials that may be available, and then arrange in some way that modifications may be made afterwards in case the particular materials expected can not be obtained. Prof. Talbot.

MR. SLATER.—The committee has been endeavoring seriously to consider the objections that have been raised here as to why the methods proposed are objectionable to the contractors, and after listening through all of the discussion, I am unable to see that there is anything running through it other than that at the present time it is impossible to judge what strength will be developed by a given concrete; therefore, do not specify any strength; specify only the mix and nobody will know what strength we have obtained. That is the way the whole discussion seems to me to line up at the present time, with the exception of that of the last speaker, and it seems to me quite evident that Prof. Talbot has in his mind some constructive suggestion. The committee would appreciate knowing as definitely as possible what he would suggest. Mr. Slater.

ERNEST ASHTON.—It is rather interesting to note that in the results given by Prof. Talbot, the average of 3,000-lb. concrete was within five per cent of the specified requirement, and the 3,000-lb. concrete within three per cent, not taking into consideration the variation, but the interpretation that might be placed on average values. Mr. Ashton.

Prof. Talbot.

PROF. TALBOT.—I do not see that the average value of the twelve different mixtures intended to give 3,000 lb. concrete is significant. If one structure were built with the proportions that gave a strength of 4,000 lb. per sq. in., and another with the mixture that produced a strength of 1,900 lb. per sq. in., the fact that the average of both buildings was near 3,000 lb. per sq. in. would not add to the stability and durability of the latter. Besides, I think there is a good deal of this averaging of test results that may be misleading.

REPORT OF COMMITTEE E-5, ON FIREPROOFING.

PRESENTED BY C. R. GOW.

After examination of all the references relating to the matter of the use of silica pebbles in concrete, we are convinced that Section B-67 which contains the expression,—"provided aggregate showing an expansion not materially greater than that of lime stone or trap rock is used," needs further discussion and consideration before it is adopted. A narrow interpretation of this paragraph would seem to make it necessary to discard all gravels containing an appreciable amount of silicious material. It must be admitted that a number of reliable tests have indicated that silicia pebbles and some other forms of silicious material are less fire-resistant as aggregates than materials like lime stone and trap rock. In some of these cases it is at least a question whether the failure is due to the large and peculiar expansion of the silica materials or whether the water contained in the minerals may not be the principal cause of disintegration.

As has already been pointed out in an earlier report, the variation in gravels, especially glacial deposits, is great from place to place even where the distances involved are not very great. Some practical means of indicating what shall be considered a satisfactory or an unsatisfactory silicious gravel, either by chemical test or by physical measurement would seem to be necessary if this paragraph is to be helpful.

From the tests made upon columns at Pittsburgh and Chicago, it is evident that the silicious materials were less effective than other forms of aggregate. It may well be that where the silicious material is used a structure will result, fireproof in a degree comparable with that afforded by other types of construction, though of lesser degree than where concrete of trap or lime stone is used. It seems probable that in some cases the superior fire-resisting qualities of concrete made with trap or lime stone will not offset the added cost. It should be borne in mind that every material has a fire-resisting limit and even assuming as we must that silicious materials will give a lower fire-resisting value than those of lime stone or trap rock, the true measure of their respective values in a structure must be measured by the insurance value of the one as compared with the other. In other words, a concrete containing silica pebbles in the aggregate will undoubtedly prove satisfactory as a fire-resistant up to a certain temperature. If it appears desirable to obtain a still higher fire-resistance, this can be accomplished by spending the added amount of money necessary to obtain lime stone or trap rock if the advantage to be gained appears to be commensurate with the cost.

It is suggested that the fitness of an aggregate for use in fire-resisting concrete may be determined by a test similar to the so-called "spalling" test used for refractory bricks, and that a material should be deemed unsatisfactory for service as an aggregate of fire-resisting concrete when it failed to pass some such simple test. This might consist in making up a number of cubes and immersing them promptly in water after heating for a stated time at a definite temperature. The loss in weight in spalling would determine the fitness of the material. Such a test would appear to be more simple and desirable than an elaborate chemical or physical test.

DISCUSSION OF COMMITTEE E-5 REPORT.

RICHARD L. HUMPHREY.—The experience with quartz gravel in the two fires where that damage was particularly great, namely, the Far Rockaway Fire and later the fire in one of the buildings (No. 41) of the plant of the Barrett Manufacturing Co., Philadelphia, convinced the speaker personally that a quartz gravel concrete that was subjected to a slow rise in temperature, would have an opportunity of adjusting itself perhaps to that temperature, whereas a fire of great intensity, rapidly developed, would produce the sparkling and shattering effect that was observed in those two structures. In the case of the Far Rockaway Fire, the burning of oil, in a relatively cold building, produced an intense fire with a rapid heating of the surface of the concrete and subsequent spalling. In the case of the Barrett plant fire the intense and rapid heat that developed from the burning naphtha caused spalling. That temperature caused actual fusing of the concrete, as was noticed in some of the adjacent beams and girders where the wire glass windows were broken and allowed the fire to pour in through the opening. Mr. Humphrey.

Tests that were made in Chicago Underwriters Laboratories, which were part of the work of the Structural Materials Testing Laboratories in St. Louis, included sections of beams made of cinder, gravel, lime stone and granite concrete. In the gravel concrete, where the quartz particles were kept away from the surface, the damage was very much less than in the case of the Far Rockaway, or Barrett Fire. The speaker feels that if the fire hazard is great and there is likely to be a very high temperature that rapidly develops, as was the case in those two fires, you can take care of the case by placing on theh exposed surface a coating of mortar of a material that spalls less than ordinary quartz gravel, that can be accomplished without any great sacrifice, keeping the particles away from the face of the exposed surface.

As the speaker understands the report, some of the granites were as unsatisfactory as trap rock. Granite has a wide range of structure. Some granites are made up of very large crystals and other granites, Dix Island granite, for instance, are made up of extremely fine grained material. In the panels in Chicago there was a piece of Dix Island granite, that had been sent to the laboratory for testing in connection with one of the federal buildings, and it was placed in the panel of natural stone and in the test this granite showed less spalling than most of the other materials. The coarse grained granite I think will spall very much like quartz pebbles. Fine grained granites are as durable as lime stones. This morning attention was called to the fact that this would rule out gravel, which is in common use in different parts of the country. There is as wide a range in the character of gravel as in granite. The glacial gravels found around Chicago, which contain large percentages of lime stone par-

Mr. Humphrey. ticles will not have the same characteristics as the pure quartz pebbles found around Far Rockaway, New York.

If a structure of reinforced concrete may have to withstand rapidly developed high temperatures, then there should be some material placed on the exterior of the exposed members that will have the requisite fire resistance. There is no reason why the core of the building should not be built of local quartz gravel.

REPORT OF COMMITTEE E-1, ON DESIGN.

PRESENTED BY A. R. LORD.

Committee E-1, on Reinforced-Concrete Design and Specifications, has consisted of fourteen members during the past year, of whom five are also members of the Joint Committee. On account of the pressing character and large amount of work that these members have had in that connection, it has not been felt advisable to assign to them much work on this committee. We will request the Board of Direction to increase our membership to eighteen during the coming year. The work of the committee during 1921-22 has included three principal items:

(a) A study of the problem of specifying concrete by strength and related matters, including factor of safety.

(b) Discussion of the technical provisions of the 1921 report of the Joint Committee.

(c) Investigation of the cost of structural members under the 1921 Joint Committee report as compared with the present cost under city building codes, excluding the matter of cost of producing concrete of any given strength.

The matter of specifying concrete by strength has been approached by this committee as a possible development of the future toward which active steps should now be taken to investigate its practicality and desirability, and to develop whatever data need be secured if it is to be equitably and safely applied. The publication of the now famous Section 28 of the 1921 Joint Committee report, has precipitated this whole question and makes the work of our committee in this report even more timely. We are not prepared to offer any suggestion as yet as to the solution of this disturbing problem. We have in fact made a bare beginning on it and it remains a major portion of our program for 1922-23 unless the formation of a special committee from committees C-6, E13, and E-1 should relieve us of this responsibility.

On our discussion of the Joint Committee report with special reference to the technical merit of its design provisions we have made progress and have contributed to the work of the Joint Committee. We feel that this work is best conducted by direct connection between our committee and the design committee of the Joint Committee and that the public discussion of the mass of suggestions and criticisms made by members of our committee would serve no useful purpose. The Joint Committee has shown itself to be receptive to our suggestions when properly supported. This work also will continue a major item of our work in the coming year. Our committee may well be used by all members of the Institute as a clearing house for their suggestions as to any part of the Joint Committee specifications. We are already in receipt of several written comments by persons not members of our committee and have found valuable material in them. We would suggest that no member may claim to know much of the significance and merits of the Joint Committee report who has not deliberately set about the task of careful criticism of its provisions and better still the still more laborious task of applying those provisions to actual designs.

Following the Am. Soc. C. E. meetings in New York in December, 1921, at which the progress report of Joint Committee was publicly discussed at great length and at which the opinion was expressed by many that the adoption of these specifications would greatly increase the cost of reinforced-concrete construction, Committee E-1 of the Institute was instructed to present a study of this question to this convention. A program was quickly drawn up and a blank form prepared calling for a large amount of design data on reinforced-concrete members together with the quantity of steel and concrete required for their construction. These were sent out to eleven well known engineers as follows:

New York, A. W. Stephens, Turner Const. Co. (A. C. I.)

Philadelphia, Manton E. Hibbs, Bureau of Buildings (A. C. I.); Maurice Webster, Wm. Steele & Sons Co.

Cleveland, Wilbur J. Watson, Consulting Engineer.

Detroit, W. S. Thompson, Corrugated Bar Co. (A. C. I.)

Pittsburgh, J. A. Ferguson, Consulting Engineer (A. C. I.).

Cincinnati, Wm. P. Anderson, Ferro Concrete Const. Co. (A. C. I.)

Chicago, Arthur R. Lord, Consulting Engineer (J. C.) (A. C. I.).

San Francisco, L. H. Nishkian, Consulting Engineer.

Each co-operator was asked to prepare an economical design as sanctioned by the code of the city and the approved practise of the building department. Similar sheets were prepared in accordance with the A. C. I. standard specifications for reinforced-concrete design (1920) and the 1921 report of the Joint Comimttee. These two sheets were prepared by Mr. Lord with the assistance of H. J. Gilkey of the University of Illinois.

Each sheet included the design and quantities for the following members, the design conditions being stated at some length to insure comparable results.

1. One one-way solid slab 12 ft. clear span, 300 lb. live load.
2. Nine beams, loads 1000 to 2000 lb. per ft., spans 16 to 24 ft. Six girders, 20,000 to 40,000 lb. loads at third pts., int. and end spans.
3. Nine flat slabs, load 100 to 300 lb. per sq. ft., spans 16 to 24 ft.
4. Six footings, load 200,000 lb. to 1,000,000 lb. soil at 4000 and 6000 lb.
5. Five spiral columns, loads 200,000 to 1,000,000 lb., 12 ft. story.
6. Four tied columns, loads 50,000 to 200,000 lb., 12 ft. story.

Each sheet represented a very large amount of work. The results to be presented include six cities and the Joint Committee and A. C. I. specifications, and cover a broad enough field to be indicative.

No attempt was made to specify beam sizes, concrete mixtures in columns, slab thickness, or other matters commonly left to the designer's judgment. All the engineers invited to participate are well versed in competitive design and are supposed to know how best to combine economy and good design under the conditions prevailing in their own city. While no claim is made that the figures for any particular city are conclusive evidence of the liberal or conservative position of the city in the matter of concrete design, still it is believed that taken as a whole these figures will show the average design cost and the general range in cost of various members used in buildings.

The committee realizes that under any given specification the cost of members may vary greatly. In Chicago for instance the cost of spiral columns may be varied from $44.40 (for an average of cases used by the committee) to over $51.00 depending on the selection of vertical and spiral percentages. The value plotted ($47.80) represents average practice and not maximum economy possible under the Chicago code.

It should be further noted in connection with these comparisons that

(a) The cost of forms is not included. It was not felt that this item would vary greatly. But the use of percentages is liable to misinterpretation, unless it is clearly kept in mind that the given costs include steel and concrete only.

(b) Costs are given for members, not for complete structures. Any saving is dead weight of the construction, which would affect the cost of other members, is not taken into account.

(c) Differences in the cost of excavation are not considered although very large variations in footing depths are included in the designs submitted.

(d) The strength of the concrete has been assumed as 2000 lb. per sq. in. (or some other given value) and the cost of 2000 lb. concrete assumed to be that of 1 : 2 : 4 concrete. This is in accord with the practice of most building departments. The question as to whether the compressive strength tests specified by the Joint Committee will require a more expensive mixture to attain this strength is not here taken into account. These figures are limited to design (the field of Committee E-1) and do not go into the very pertinent question of strength of concrete. It is undoubtedly true that the higher stresses permitted by the Joint Committee are based upon the assurance that a 2000 lb. concrete will be secured by these specifications. If it be true that a 2000 lb. concrete cannot be produced at the same price as ordinary 1 : 2 : 4 concrete, the showing of these figures must be modified by other figures not now at hand showing the cost of a Joint Committee 2000 lb. concrete as compared with ordinary 1 : 2 : 4 concrete. The quantities of concrete given in Table I will permit this extension of these data to be made readily when such additional cost data is at hand.

(e) Bond provisions have been disregarded in large measure, since they are commonly not strictly enforced by the city building departments and to apply them in the Joint Committee and A. C. I. figures would add an unbalanced cost.

(f) Undoubtedly other aspects in which this comparison is limited and indicative only could be mentioned and will be brought out in the discussion.

The cost of concrete and steel as given by the assisting engineers for their various cities varied considerably. The average values used in this comparison were as fellows:

2000 lb. concrete in place in forms	$9.50	per cu. yd.
2500 lb. “ “ “ “ “	11.00	“ “ “
2900 lb. “ “ “ “ “	12.50	“ “ “
3000 lb. “ “ “ “ “	12.80	“ “ “
Reinforcing steel, bent and placed	62.00	per ton
Column vertical bars, in place	60.00	“ “
Column spiral, in place	78.00	“ “

These figures are intended to cover contractor's profit and all job expense commonly charged against the concrete work. In as much as quantities of materials are given in Table I, modified costs for varying unit prices in different cities may be readily computed. Different unit prices will change the resulting comparison materially for some members.

Not much attention was paid to one-way solid slab designs for the reason that slab thicknesses are commonly determined by judgment in beam and slab structures and the theoretical slab thickness as found by the permitted stresses is rarely used. It would be too thin. Hence the apparent difference in cost as shown in Table I for one-way solid slabs applies only to rather exceptional cases.

The designs in accordance to Joint Committee and A. C. I. specifications were made after five sets of city figures were at hand and it was possible to use average sizes of members. None of the various co-operators felt like making a Joint Committee design after having put in the necessary work on their own city code design so the figures here presented for the Joint Committee and A. C. I. are single values and not averages of values secured by different engineers. The various computations have not been checked by the committee. The committee has simply collected the data into one table and six diagrams and presents these for the information of the Institute in its consideration of the Joint Committee report of 1921.

A study of Fig. 1 to 6 and Table I is conclusive as to the needs of uniformity in building codes—one primary object of the Joint Committee work. The values vary tremendously for the different cities.

The design and costs of individual members as given thus far is essential to the critical comparison of the specifications but a better general view may be obtained by preparing figures for a complete building and showing the cost comparison on this basis. To this end we have taken the following distribution of cost from figures prepared by the New York contractors' committee for a building 80 x 300 ft., 6 stories, and averaging live-loads usually employed. Each percentage represents the proportionate part of the total cost attributable to each type of structural member.

	Flat-Slab Building	*Beam and Girder Building*
Floors and Roof	65%	68%
Tied Columns	8%	9%
Spiral Columns	12%	11%
Footings	13%	10%
Walls, etc. (of concrete)	2%	2%
Total	100%	100%

Table II has been prepared from Table I and these percentages represent the ratio of cost under Joint Committee provisions to the cost on the basis outlined in preceding paragraphs for the various cities and the A. C. I. specifications of 1920. It should be remembered that these figures are but rough approximations for any given city but the average is probably quite indicative of the nation wide situation. They should be studied rather than memorized.

Table II.—Rates of Joint Committee to Other Specifications.

Code or Specification	*Flat-Slab Building*	*Beam and Girder Building*
Am. Conc. Inst. 1920	1.09	0.90
Chicago	1.03	0.87
Philadelphia	1.02	0.74
Cleveland	0.98	0.68
New York	1.11	1.00
Detroit	1.13	0.98
Cincinnati	1.24	1.10
San Francisco	1.02	0.91
Average of eight	1.08	0.90

These figures are for the cost of the steel and concrete only. To get at building costs we must add the approximately constant costs of excavation, brickwork, mill work, plumbing, electrical work, heating, roofing, formwork, etc., etc. The costs covered in the above figure may be taken to represent roughly 20% of the cost of the completed buildings and if so taken the average figures are reduced to

1.02 for flat slab building
0.98 for beam and girder building.

Even if the cost of the concrete and steel were considered as great as 30% of the entire cost of the building these figures would be

1.02 for flat slab building
0.97 for beam and girder building.

TABLE I.—RATES OF JOINT COMMITTEE TO OTHER SPECIFICATIONS.

Kind of Member.	No. of Values.	Material.	Unit Cost.	Joint Committee Specification 1921.		American Concrete Institute Specification 1920.		Chicago.		Philadelphia.		Cleveland.		Detroit.	
				Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.
One-way Solid Slab.....	1	2000 lb. conc......	$9.50	70.5	$24.80	78	$27.50	81	$28.50	92	$34.20	75.5	$26.60	81.5	$28.70
	..	Reinf. bars........	62.00	345	10.70	386	12.00	344	10.70	382	11.90	380	11.80	325	10.10
Total Cost.........	..	....	.....	.....	$35.50	.....	$39.50	.....	$39.20	.....	$46.10	.....	$38.40	.....	$38.80
Tee Beams and Girders..	15	2000 lb. conc......	$9.50	33	$10.60	43	$15.10	42.5	$15.00	46	$16.20	52.5	$18.50	38	$13.40
	..	Reinf. bars........	62.00	290	9.00	336	10.40	276	8.60	382	11.90	488	15.10	256	7.90
Total Cost.........	..	....	.....	.....	$19.60	.....	$25.50	.....	$23.60	.....	$28.10	.....	$33.60	.....	$21.30
Flat Slabs............	9	2000 lb. conc......	$9.50	326	$115.00	324	$114.20	322	$113.40	294	$103.50	306	$107.80	308	$108.60
	..	Reinf. bars.......	62.00	1440	45.60	1146	35.50	1140	35.30	1220	37.80	1460	45.20	965	29.90
Total Cost.........	..	....	.....	.....	$160.60	.....	$149.70	.....	$148.70	.....	$141.30	.....	$153.00	.....	$138.50
Footings...............	6	2000 lb. conc......	$9.50	235	$82.80	225	$79.20	343	$120.80	480	$169.00	423	$149.00	394	$138.80
	..	Reinf. bars.......	62.00	921	28.50	1150	35.60	577	17.90	934	29.00	753	23.40	671	20.80
Total Cost.........	..	....	.....	.....	$111.30	.....	$114.80	.....	$138.70	.....	$198.00	.....	$172.40	.....	$159.60
Spiral Columns.........	5	2000 lb. conc......	$9.50	.....	.....	6	$2.10	.....	.....	16.5	$5.80	47.5	$16.70	.....	.....
	..	2500 lb. conc......	11.00	.....	.....	7.5	3.10	.....	.....	46	18.80	.....	.....	.....	.....
	..	2900 lb. conc......	12.50	38	$17.60	33.5	15.90	63.5	$29.40	.....	.....	.....	.....	.....	.....
	..	3000 lb. conc......	12.80	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	56	$26.60
	..	Vertical bars......	60.00	830	24.90	421	12.60	306	9.20	564	16.90	913	27.40	166	5.00
	..	Spiral Steel.......	78.00	166	6.50	283	11.00	236	9.20	268	11.40	200	7.80	284	11.10
Total Cost.........	..	....	.....	.....	$49.00	.....	$44.70	.....	$47.80	.....	$52.90	.....	$51.90	.....	$42.70
Tied Columns..........	4	2000 lb. conc......	$9.50	5.5	$1.90	3	$1.10	5.5	$1.90	28	$9.90	19	$6.70	3	$1.10
	..	2500 lb. conc......	11.00	6.5	2.70	4	1.60	6.5	2.70	.....	.....	.....	.....	.....	.....
	..	2900 lb. conc......	12.50	18.5	8.60	12	5.70	18.5	8.60	.....	.....	.....	.....	.....	.....
	..	3000 lb. conc......	12.80	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	14.5	6.90
	..	Reinf. steel.......	62.00	76	2.40	66	2.10	78	2.40	190	5.90	306	9.50	62	1.90
Total Cost.........	..	....	.....	.....	$15.60	.....	$10.50	.....	$15.60	.....	$15.80	.....	$16.20	.....	$9.90

TABLE I. (Continued).—RATES OF JOINT COMMITTEE TO OTHER SPECIFICATIONS.

Kind of Member.	No. of Values.	Material.	Unit Cost.	Joint Committee Specification 1921.		American Concrete Institute Specification 1920.		Cincinnati.		San Francisco.		Pittsburgh.		New York.	
				Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.
One-way Solid Slab	1	2000 lb. conc.	$9.50	70.5	$24.80	78	$27.50	89.5	$31.50	70	$24.60	92	$32.40	89.5	$31.50
	..	Reinf. bars	62.00	345	10.70	386	12.00	262	8.10	30	9.30	369	11.40	348	10.80
Total Cost	..	.....	.....	.....	$35.50	.....	$39.50		$39.60		$33.90		$43.80		$42.30
Tee Beams and Girders	15	2000 lb. conc.	$9.50	33	$10.60	43	$15.10	34	$12.00	38.5	$13.60	97	$33.50	34.1	$12.00
	..	Reinf. bars	62.00	290	9.00	336	10.40	220	6.80	282	8.80	105	3.30	329	10.20
Total Cost	..	.....	.....	.....	$19.60	.....	$25.50	.....	$18.80	.....	$22.40	.....	$36.80	.....	$22.20
Flat Slabs	9	2000 lb. conc.	$9.50	326	$115.00	324	$114.20	287	$101.10	316	$111.30	.....	.....	313	$111.60
	..	Reinf. bars	62.00	1440	45.60	1146	35.50	870	27.00	1316	40.80	.....	.....	1263	39.10
Total Cost	..	.....	.....	.....	$160.60	.....	$149.70	.....	$128.10	.....	$152.10	.....	.....	.....	$150.20
Footings	6	200 lb. conc.	$9.50	235	$82.80	225	$79.20	209	$73.60	434	$152.80	.....	.....	234.5	$82.50
	..	Reinf. bars	62.00	921	28.50	1150	35.60	629	19.50	660	20.50	.....	.....	754	23.40
Total Cost	..	.....	.....	.....	$111.30	.....	$114.80	.....	$93.10	.....	$173.30	.....	.....	.....	$105.90
Spiral Columns	5	2000 lb. conc.	$9.50	.....	.....	6	$2.10	.....	.....	.....	.....	.....	.....	.....	.....
	..	2500 lb. conc.	11.00	.....	.....	7.5	3.10	65.5	$26.70	.....	.....	.....	.....	47.7	$19.40
	..	2900 lb. conc.	12.50	38	$17.60	33.5	15.90	.....	.....	.....	.....	.....	.....	.....	.....
	..	3000 lb. conc.	12.80	.....	.....	.....	.....	.....	.....	55	$26.10	.....	.....	.....	.....
	..	Vertical bars	60.00	830	24.90	421	12.60	325	9.80	254	7.60	.....	.....	207	6.20
	..	Spiral Steel	78.00	166	6.50	283	11.00	240	9.40	194	7.60	.....	.....	389	15.20
Total Cost	..	.....	.....	.....	$49.00	.....	$44.70	.....	$45.90	.....	$41.30	.....	.....	.....	$41.80
Tied Columns	4	2000 lb. conc.	$9.50	5.5	$1.90	3	$1.10	7	$2.50	29	$10.20	.....	.....	20.6	$7.20
	..	2500 lb. conc.	11.00	6.5	2.70	4	1.60	12	4.90	.....	.....	.....	.....	.....	.....
	..	2900 lb. conc.	12.50	18.5	8.60	12	5.70	.....	.....	.....	.....	.....	.....	.....	.....
	..	3000 lb. conc.	12.80	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	..	Reinf. steel	62.00	76	2.40	66	2.10	104	3.20	135	4.20	.....	.....	94	$2.90
Total Cost	..	.....	.....	.....	$15.60	.....	$10.50	.....	$10.60	.....	$14.40	.....	.....	.....	$10.10

The committee does not desire at this time to discuss these results or to criticize the Joint Committee report which we understand is still under process of construction and revision. Our committee is so numerously represented on the Joint Committee that we have no difficulty in securing the most thorough and influential consideration for any suggestions we may have to make as to the detailed provisions of the design sections. This committee does feel that the present Joint Committee is approaching its task in a spirit that bids fair to make its findings acceptable to the Institute and to the profession generally.

The speaker wishes in conclusion, to make it perfectly clear that he is speaking in the capacity of chairman of A. C. I. Committee E-1, and not as a member of the Joint Committee.

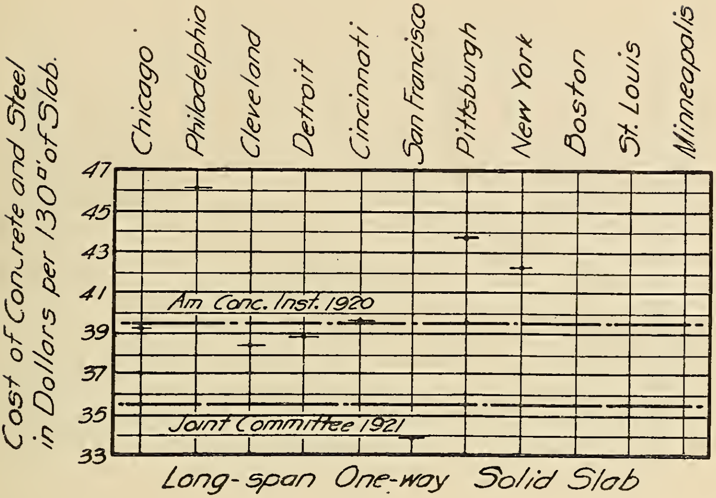
Cost of Concrete and Steel in Dollars per 130 □' of Slab.
Chicago
Philadelphia
Cleveland
Detroit
Cincinnati
San Francisco
Pittsburgh
New York
Boston
St. Louis
Minneapolis
47
45
43
41
39
37
35
33
Am. Conc. Inst. 1920
Joint Committee 1921
Long-span One-way Solid Slab

FIG. 1.

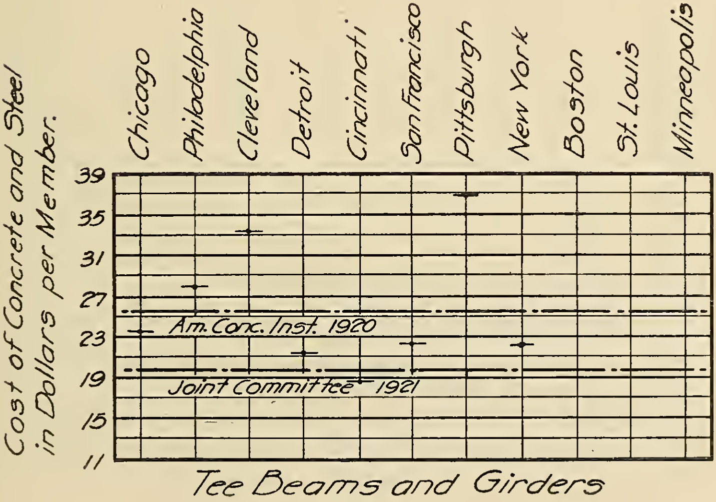
Cost of Concrete and Steel in Dollars per Member.
Chicago
Philadelphia
Cleveland
Detroit
Cincinnati
San Francisco
Pittsburgh
New York
Boston
St. Louis
Minneapolis
39
35
31
27
23
19
15
11
Am. Conc. Inst. 1920
Joint Committee 1921
Tee Beams and Girders

FIG. 2.

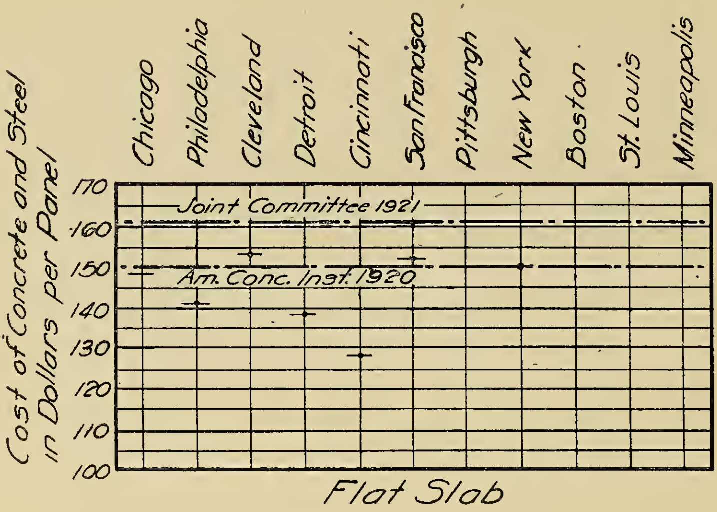
Cost of Concrete and Steel in Dollars per Panel
Chicago
Philadelphia
Cleveland
Detroit
Cincinnati
San Francisco
Pittsburgh
New York
Boston
St. Louis
Minneapolis
170
160
150
140
130
120
110
100
Joint Committee 1921
Am. Conc. Inst. 1920
Flat Slab

FIG. 3.

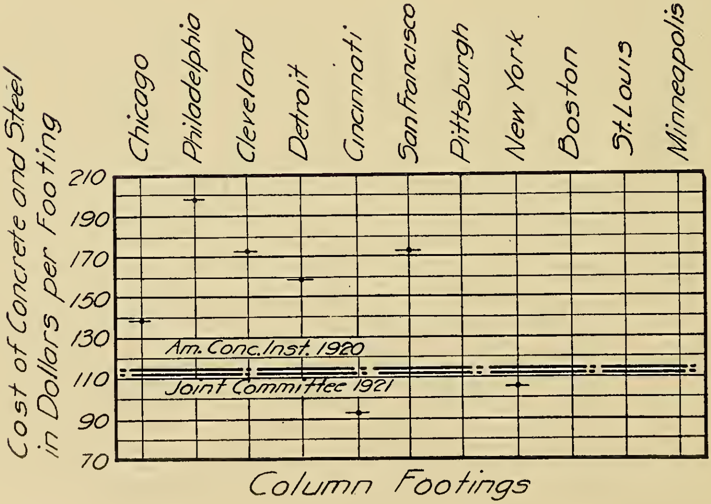
Cost of Concrete and Steel in Dollars per Footing
Chicago
Philadelphia
Cleveland
Detroit
Cincinnati
San Francisco
Pittsburgh
New York
Boston
St. Louis
Minneapolis
210
190
170
150
130
110
90
70
Am. Conc. Inst. 1920
Joint Committee 1921
Column Footings

FIG. 4.

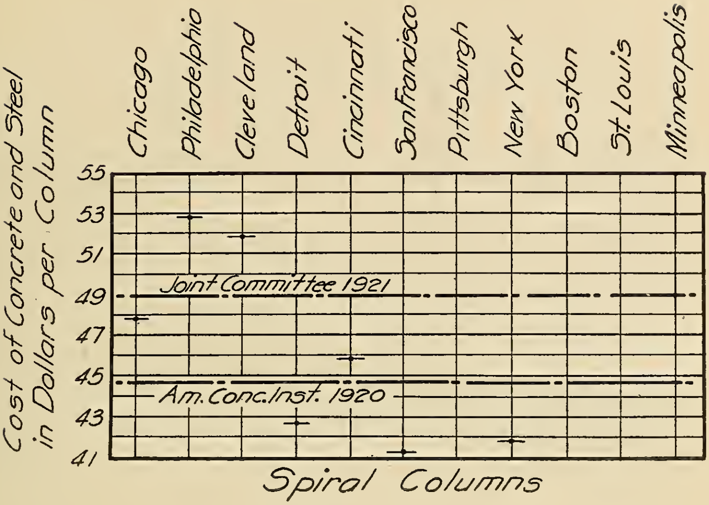
Cost of Concrete and Steel in Dollars per Column
Chicago
Philadelphia
Cleveland
Detroit
Cincinnati
San Francisco
Pittsburgh
New York
Boston
St. Louis
Minneapolis
55
53
51
49
47
45
43
41
Joint Committee 1921
Am. Conc. Inst. 1920
Spiral Columns

FIG. 5.

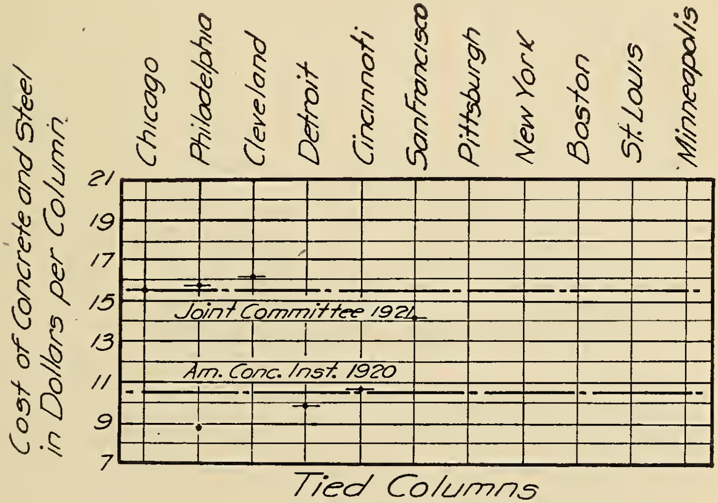
Cost of Concrete and Steel in Dollars per Column
Chicago
Philadelphia
Cleveland
Detroit
Cincinnati
San Francisco
Pittsburgh
New York
Boston
St. Louis
Minneapolis
21
19
17
15
13
11
9
7
Joint Committee 1921
Am. Conc. Inst. 1920
Tied Columns

FIG. 6.

MISCELLANEOUS SUBJECTS.

CONTRACTION JOINTS, ETC.

Mr. Turner. CHAIRMAN H. C. TURNER.—The next subject is contraction joints and expansion joints. Is there any discussion? From a number of sources I have heard criticism made of Section 75, which requires expansion joints in buildings if they are in excess of 200 ft. in length and where the width is less than half the length. To many this has seemed to be a limitation which was unnecessary. Many buildings have been constructed of much greater length than 200 ft. without expansion joints.

Mr. Wason L. C. WASON.—I think this is an unnecessary limitation of length. I know in the early days of reinforced concrete, designers considered it a necessity to tie a building together so that it would not crack from end to end, and I have executed buildings from the designs of others which ran as high as 450 ft., all beams and exterior being so reinforced continuously as to exceed the shrinkage of the concrete and therefore prevent its cracking. I think that is thoroughly first class construction. One particular building has been up for fifteen years, at least. I have nct seen it for the last five years, but at the end of ten years after construction no cracks had appeared. Presumably there were a few there, but they were so fine and so many of them because of the reinforcement resisting their cracking at one place, that they did not show. I think that 200 ft. is a very short length and that in specifying that absolutely it shall be no more than that, it should be made more elastic.

Mr. Ferguson. J. A. FERGUSON.—In Section 71 there is no statement that construction joints should be as near as possible normal to the lines of force at which the joint is made. I have seen entirely too many cases where joints at the center of spans or point of maximum compression are 45 deg. when they should have been vertical, and at the third point they were vertical when they should have been 45 deg. Something should be placed in this section to take care of that point.

WATERPROOFING AND PROTECTIVE TREATMENT

Mr. Turner. CHAIRMAN H. C. TURNER.—We will pass on to Waterproofing and Protective Treatment. Is there any discussion?

Mr. Humphrey. RICHARD L. HUMPHREY.—There is quite a variance in the committee in reference to the treatment of waterproofing. The committee has not reached any final conclusions as to whether or not to treat waterproofing in the report. Probably those things that may be added to the concrete should be dealt with; whether anything else should be touched upon I am not prepared to say. This represents as far as the committee reached any conclusions after having considered the subject extensively, and they boiled it down to this in the tentative stage.

Mr. Rhett.

A. H. RHETT.—The Tentative Specification contains some very valuable information, but devotes small space to the very large subject of waterproofing. The recommendations though brief are pointed. Regardless of practical considerations we are told that (1)—Integral Compounds shall not be used and (2)—Membrane Waterproofing shall be used, etc. The speaker acknowledges to a feeling of the greatest astonishment in reading these recommendations and finds that there are many others who share this feeling.

He holds no special brief for integral waterproofing, as his original interest in the subject of waterproofing was aroused through the necessity of waterproofing a large number of bridge structures of various types being designed and constructed under his supervision, and in connection with which work he developed some very violent theories on the subject of membrane waterproofing, which he applied with apparent success. His feeling on the subject of integral waterproofing was skeptical enough to have suited the most violent antagonist and his first use of the material was dictated by curiosity. The results obtained, however, most certainly did not seem to warrant any such dictum as that "integral compounds shall not be used," but on the contrary that there was subject matter for serious study and this study has confirmed the correctness of that judgment and resulted in some very definite conclusions.

In the first place, the generally accepted opinion that there is a direct conflict between the integral and membrane methods and that it is simply a matter of personal choice, does not seem to be substantiated by consideration of the facts involved.

Thin concrete sections subject to wide temperature variations, vibration or impact, will inevitably crack and the waterproofing problem is not one of correcting absorption or permeability, but of waterproofing cracks. Obviously, the only way to effect this result is to protect the cracks with a membrane or coating elastic enough to span these cracks when open and waterproof them. On the other hand, occasion arises, such as in a deep foundation and wet excavation, where a waterproofing is required, and yet where the constant presence of water or conditions of excavation preclude the use of a membrane because a membrane cannot be applied to a wet or unprepared surface. In such instances as there would be no temperature variation and mass concrete would probably be used, it would be obviously desirable to use integral waterproofing. Furthermore, it is being recognized more and more every day that concrete is not permanent unless waterproofed, any more than steel or wood, and here again cases arise where surface coatings are not practical and where intergral waterproofing could be advantageously used, if there be such a thing as integral waterproofing.

The committee has evidently concluded that there is no such thing, while the writer, after a careful and prolonged study, has concluded that there is and, furthermore, that the theories on which some at least of the materials are produced are technically rational and correct.

Mr. Rhett. In any effort to render concrete or mortar intrinsically waterproof, two factors must be borne in mind, and these are that water enters in two ways,—by absorption through minute pores and by percolation through pores of appreciable size. This difference in action can be very easily shown by trying to force water through a neat portland cement sample and a 1 : 2 : 4 (say) concrete sample. In the first no water could be forced through, while in the second probably considerable. On the contrary, if both are immersed in quiescent water, the neat cement sample will absorb much more than the concrete one.

Taking cognizance of these facts, waterproofing materials seem to have been produced mainly on two theories which appear diametrically opposed at first sight, but not so much so on closer examination. These are (1)—Water repellent materials and (2)—Water absorbent and lubricating materials. The first are designed to kill absorption, the second to eliminate porosity.

As far as mortars and stucco are concerned, the first theory seems entirely rational, because as the mortar consists of finely particled materials, applied under pressure, there are no real voids and water enters chiefly through absorption.

As regards concrete there is, however, room for difference of opinion. The water-repellent theory assumes that if the mortar is rendered non-absorptive, the concrete is waterproof because the coarse aggregate does not absorb. The lubricating and water absorbent theory, going more boldly at the subject, believes that as the mortar is but a small part of the volume, absorption is a minor factor and that the elimination of permeability through the elimination of porosity is the end to be accomplished. Porosity is due largely to faulty field manipulation; that is, to too much water, to little tamping; to vertical chuting, insufficient mixing, segregation of material and insufficient absorption of the water by the cement. If the lubricating element can be introduced, the liability to defects from superfluous water, insufficient tamping and vertical chuting is certainly lessened, to say the least.

While if absorption of the water by the cement can be increased and this accompanied by a slight swelling action, the liability to porosity from the evaporation of unabsorbed water is lessened.

The skeptic will no doubt say that this is all very well in theory, but why can it not be proved, by which he means apparently, proved by laboratory experiment. The writer submits with all due respect that this may be due to defective experiments and not defective materials. It is possible to test in a laboratory the absorption killing effect on mortars, but nullification of field errors cannot be tested until laboratory experiments can reproduce the field errors. It is difficult to see, too, on what basis of reasoning the result of a laboratory experiment can be considered more convincing that results obtained in actual field operations. Most certainly actual results obtained in field operations have convinced a very large number of practical users of the value of integral compounds when properly used.

As regards laboratory tests, however, no really authentic and comprehensive tests have ever been made as far as the writer's knowledge goes. Much has been made of the tests conducted by the Bureau of Standards in 1911 and their generally assumed adverse findings. As carefully as these tests were made, a detailed examination, which many have apparently failed to make, does not substantiate this assumption. For instance, on page 64 the following statement occurs, referring to the results of the tests as applied to a 1 : 4 sand mortar, after four weeks' immersion: "It will be observed that all of the mortars containing water-repellent compounds were damp-proof." And Table 19 shows further that while the untreated specimens failed after 58 days, three specimens containing water-repellent compounds were sound after respectively 110, 136 and 164 days.

It would also no doubt surprise many of the constant quoters of this report, to know that the tests did not even include concrete, but were made solely on mortars. In no other factor pertaining to good concrete would it be considered valid to reason from mortar to concrete in view of today's knowledge, and certainly, as it was attempted to show above, it is not valid to do so in regard to water-proofing.

It is the writer's conclusion that this whole subject has been clouded and obscured by a lack of comprehension and that if this obscuration can be cleared away, much of the antagonism would also disappear.

It can no longer be disputed that the entrance of water into concrete is destructive of its permanence and means must be produced to exclude it and that the means suitable for one condition may not be applicable for another.

It is to be assumed that the labors of this committee are directed toward obtaining the best concrete possible for every condition that may arise and most certainly this end is not furthered by closing the door in the face of effort to obtain that result along whatever line it may be expected.

Mr. Humphrey.

Richard L. Humphrey.—A good deal has been said about the chapter on waterproofing. Mr. Wason has said that he knows of integral waterproofing compounds that have been used advantageously. It seems to me that if the concrete cracks, the mere fact that integral waterproofing material was used is not going to stop leakage at the crack. The committee has the belief that if concrete is properly proportioned and mixed, it can be made sufficiently dense to prevent the passage of water. In Philadelphia, during the construction of the subway, there was torn out around the City Hall a granolithic pavement laid about 1882; it had ¼ in. reinforcing rods, such as it was the practice to use at that time; in a number of cases the rod was within ¼ in. of the surface, where it was exposed to the weather, and those rods came out perfectly clean without any appearance of rust.

The proponents of the integral compounds are making statements as to their value; if they would reverse their position and put themselves in the position of the members of this committee and take the published

Mr. Humphrey. tests and balance the results of those tests against the claims of those who have these compounds for sale, they would understand why the committee reached the conclusion given in the report. Mr. Rhett speaks about the tests in St. Louis published in Technologic Paper No. 3. At the time that series of tests was undertaken, there was a widespread use of machines for making cement building blocks; in order to make cheap blocks, it was the practice to use one part cement and as high as ten parts of sand, and it was also the practice to use waterproofing to overcome the defects of badly proportioned mortars and make them stand up even in ordinary weather exposures. Now it was at that time that the Structural Materials Testing Laboratories at St. Louis purchased in the open market waterproofing compounds all over the country and conducted those tests. The tests were made on lean mortar for the purpose of testing the efficacy of these waterproofing compounds or any new ones. Concrete was also tested in connection with that series of tests. The men who had charge of those tests, including the speaker, had no proprietary interest in the tests and was ready to see any virtue in the waterproofing materials used. The unmistakable conclusion of those tests was that the same or better results could be obtained by the use of the ordinary ingredients of concrete properly proportioned and mixed. Now it seems to the speaker that if those who believe that waterproofing materials are desirable will devote their energy to submitting to the committee the proof of it in supporting tests, they will be helping their cause a great deal more than by merely telling about it in words.

GENERAL DISCUSSION.

W. A. SLATER.—There are two questions I want to ask for information for the use of the Joint Committee. The first alternate of Section 28 has as its first sentence a statement which I believe is the source of part of the misunderstanding or part of the misconception as to the results of this specification. This section starts as follows: "The proportions of cement, water and aggregate shall be such as to produce concrete of the strength and quality specified in sections three and four"; and then the following: "The proportions shall be one part of portland cement, —— parts of fine aggregate, etc." The indication seems to be that the committee is specifying both the strength of the concrete and the proportions, and it seems quite likely that it is on that feature that has hinged a great deal of the discussion. It was not the intention of the committee to specify both strength and proportions, and the question I wish to ask is whether, if this section were to be modified, either by the omission of the first sentence or by some other provision which would make it clear that it does not specify both strength and proportions, the remedy would make the first of the alternates of Section 28 acceptable to those who have most objected to it? Mr. Slater.

My second question has to do with the second alternate of Section 28. That section provides for the furnishing by the contractor of concrete of a given strength, and lays down no specifications as to how the concrete shall be proportioned in order to obtain that strength. Undoubtedly there are jobs in which there is no engineer independent of the contractor, and if it were left to the engineer to see that the proper proportions and proper strength were obtained, it would be simply putting it up to the contractor after all. In order to take care of those cases where there is no independent engineer, would it be satisfactory to the contractors who are here represented if a clause were introduced into this section which specifies that in cases where no engineer independent of the contractor is employed, the contractor shall use materials so proportioned and mixed as to produce concrete of the required workability and strength, the remainder of the section presumably reading as it now stands.

DEF. H. DIXON.—I would like to ask one question, first, of Mr. Slater, and that is this: alternate 2 provides for a specification of concrete for strength; alternate 3 provides for specification by proportions; therefore, I do not understand what the idea is in alternate 1. If alternate 1 does not mean to specify concrete by both strength and proportions, what does it mean? Because in 2 and 3 you covered the two separate cases. Mr. Dixon.

PROF. SLATER.—I did not realize that there was any question as to the further meaning of the first alternate of Section 28. The purpose of that section is to require in cases where that alternative is selected that the proportions shall be specified rather than the strength, but that those proportions shall be based upon tests which have previously been made Mr. Slater.

Prof. Slater. to determine what strength may be reasonably expected from the materials to be used. If it does not express that idea the committee should have the fact brought out and should make the necessary changes.

Mr. Dixon. MR. DIXON.—Yes, but the point is that at the present time in Section 4 you state what strength in concrete is as soon as you refer to that; you do specify both strength and proportions.

Mr. Slater. MR. SLATER.—Apparently some things we supposed were understood are not clear. The distinction between the three alternates of Section 28 as intended is as follows: The first section specifies proportions based upon strengths found from tests made previous to the beginning of the work.

Prof. Talbot. PROF. TALBOT.—Or before the letting of the contract. I have asked that question several times and never got the same answer. There is nothing definite here as to whether this is to be filled out before the contract is let. I cannot see how it could be filled out afterwards with any justice.

Mr. Humphrey. MR. HUMPHREY.—That is the understanding. It is to be filled in by the engineer before the contract is let.

Prof. Talbot. PROF. TALBOT.—Others have said you have got to wait until you find what material is going to be used; this contractor intends to use this material and that contractor intends to use that, and after the contract is left, the material is to be determined.

Mr. Lord. MR. LORD.—If Mr. Slater will yield the floor for just a moment, I think it is the general understanding of the Joint Committee itself that the first alternate of Section 28 is intended to apply to a large building where it is possible for the engineer to go to the expense of making proper tests, and from the results of those tests specifying his material by proportions in advance of the contract. The last alternate of Section 28 is intended to apply to a very small structure where such a specification is not warranted and where the engineer is forced to specify arbitrarily without such tests what mixtures shall be used where the tables are relied on, and some substitute for those tables must be found to make it workable for small buildings.

Mr. Dixon. MR. DIXON.—As I understand Mr. Slater, the intention of the committee was to make alternate 1 cover the case where the engineer makes tests before he writes his specifications and fixes the strength in that way, and that alternate 3 was left for a case where he did not have time to make tests and he went to Table IV. Now so far as the purpose of the specification is concerned I do not see any distinction there myself. The specification by proportioning covers both those cases; it is merely that in one case the engineer makes some tests first and establishes it himself, and in the other case he goes to the table. I do not think myself that in a standard specification reference should be given to a table to establish the strength of the concrete. Certain engineers might feel that was right and others might feel it was not right. You have got two ways of specifying concrete: one is by strength, putting it up to the contractor, and the other is by proportion, and I cannot see the necessity for the first alternate and a second alternate to cover the slight distinction. If you want to say that under the specification by proportion the engineer can make his tests himself, or if he cannot, he can go to a table, all right, but I do not think it is necessary to write a table for your specifications. When you do it becomes confusing, as in this case. Mr. Dixon.

D. A. ABRAMS.—Mr. Dixon seems to feel that a table in a specification is out of place. As a matter of fact, the Iowa State Highway Department for the past year or more has a table in its specification which provides a sliding scale of mixtures based on the proportion of sand in the gravel. That is not an impossible thing; it works, and there is no difficulty about it so far as I can see. The contractor knows exactly what he is bidding on; there is no mystery about it and it provides a remedy, a method whereby pit run gravels which are clean can be used with entirely satisfactory results. Mr. Abrams.

RICHARD L. HUMPHREY.—Table IV is not a part of the specification; it is put in for convenience just as reference is made to the specifications of the American Society for Testing Materials. They may be printed in the appendix. This table aids the engineer who has not time to make tests, in assuming what proportions he is going to write into the specifications. Mr. Humphrey.

N. M. LONEY.—Mr. Slater asked a definite question. We hold that a reading can be made of this first paragraph to make it mean that the contractor sells concrete by strength. If that can be taken out, there is no objection to it whatever. Mr. Loney.

MR. LORD.—We have an answer now to the first question from one of the contractors; I do not know whether the others would care to answer it differently. There was a second question which it might be worth while to have read. Mr. Lord.

MR. SLATER.—The second question was as to the second alternate of Section 28 which specifies merely the strength of the concrete without specifying the method by which it is to be obtained. If that section were changed so as to make it apply only to those cases where there is no engineer who is independent of the contractor, would that section then be satisfactory to the contractor? This is for the case where there is no engineer to settle the questions which, according to several who have discussed these subjects, must be put up to the engineer. Mr. Slater.

MR. LONEY.—I would like to answer that. As a man who formerly wrote specifications on which buildings were constructed, I think that is putting something into print which is utterly unworkable. You are flying directly in the face of any code which may exist, and I cannot imagine the relation between the contractor and owner where there is somebody representing the owner who would have to act as engineer or architect, at least nominally, for the purpose of the contract, so that it seems to me that such a clause could not possibly be inserted in an engineering document, if you want to call it such. Mr. Loney.

Prof. Slater.

PROF. SLATER.—I think Mr. Loney has gotten at the root of the thing in that expression, "at least nominally for the purpose of carrying out the contract." If no independent engineer is employed the "nominal" engineer is really the contractor who virtually represents the owner, and we fool ourselves if we place responsibility on the engineer when there is no responsible engineer.

Mr. Loney.

MR. LONEY.—Practically that specification would mean nothing then, and in such a case the owner places himself entirely in the hands of the contractor, and the specification is useless.

Mr. Turner.

H. C. TURNER.—I do not think that we are getting anywhere with this discussion. There are not two sides to the question of reinforced-concrete structures. As long as we have had reinforced concrete, contractors have been quite as much interested as engineers in the promotion of better designs and better construction, and the division which seems to occur now is simply over two propositions, one where the engineer may want to draw more clearly a line of responsibility. Certainly the contract under the altered specification would impose a responsibility on the contractor which does not exist in other forms of contracts where the material is to be furnished by the contractor. Quality cannot be determined at the time of manufacture, it has to be determined at some future date, and along with that there is great uncertainty which is recognized by everyone here, the great uncertainty as to the strength you would derive by the cylinders when tested at 28 days. We have an uncertainty in cement which at 28 days we have reason to think is 100%; that one thing is of itself sufficient for us to pause in any consideration or plan to adopt a specification of this character. We certainly must pause long enough to determine why there should be this variation in the strength of concrete due to the one element of cement.

Then we have the other element of a want of greater care in aggregates and in the personal equation of the making of the cylinders and testing them. It would seem that until the art had progressed further with the application to jobs of laboratory research work—until the time has come when we can forecast much more clearly than we can now, we ought not to attempt to bring out a specification of this kind. There is no feeling on the part of the contractors' committees which have reported here, who were appointed by the President. They have diligently studied the problem without bias, without any preconceived notions; they have studied it realizing that upon them would come certain obligations of a very serious nature. Properly considered, at the same time, their motive is a motive which ought not to be questioned and is not questioned.

Mr. Humphrey.

RICHARD L. HUMPHREY.—The speaker would like to suggest that the committee is deeply grateful for all the suggestions and criticisms that have been offered as to this specification. It was put forward with that end in view, and he would ask those who feel that this specification is not right, instead of criticising, to send to the committee something constructive which they think will improve the specification, if section 28, for example, is not satisfactory, will those who object to it write a section 28 which they think would be more satisfactory? **Mr. Humphrey.**

Mr Godfrey. Edward Godfrey *(by letter)*.—Several members of the Joint Committee have assured me that some of my ideas were incorporated in the new report. The new report does in fact allow the only kind of shear reinforcement for beams that can be analyzed and that will definitely and positively carry the end shear, the only kind of reinforcement for shear that has no failure to its credit, the kind of reinforcement that I have been fighting for for fifteen years. I refer to bent up and anchored main rods. Not a patented or proprietary thing but just the most natural and rational reinforcement imaginable.

But even this type of reinforcement the report throttles and almost nullifies by an absolutely foolish and senseless limitation of the angle of the slope. Because, in the judgment of the committee, the angle should be limited to 20 deg. with the horizontal, this limitation is set. Tests made on a finished building by Messrs. Talbot and Slater (Illinois Experiment Station, Bulletin No. 64) show excellent results on bent up and anchored rods flatter than this report allows—high stress in the rods and not a crack in the beams. The same bulletin describes tests on stirrup installations and bent up and not anchored rods. The stirrups and unanchored rods were in compression, and the beams were cracked. Yet they ask me to produce evidence, facts to prove my contention. In the name of all that is true, what better evidence could I hope for or produce than these results of tests made by the very men who have opposed my stand and fought against it for so many years?

This new report administers oxygen to the stirrup or short shear member in a desperate effort to keep it alive a few years longer. Stirrups must now be anchored to top and bottom steel, and the top steel must be anchored to the supports. Thus continuity of shear reinforcement and end anchorage of the same are at last recognized. These are the things I have fought for so long. They are both effected by the use of bent up and anchored rods, and the latter do it in a manner that is vastly superior to the stirrup system.

But there are anchors and anchors. A stirrup with a fish-hook bend around a horizontal rod, as described by this report, is anchored in name only and not in fact, and even with all of this, if the stirrups are vertical, we shall have a system where the stirrups cannot act until, as Prof. Talbot has demonstrated, the concrete web has failed.

One of the worst features of the present report is the exceedingly high unit shear allowed on beams. The unit stress of 240 lb. per sq. in., allowed in this report, is acknowledged by Mr. Slater to be about the unit stress at which his I-beam tests began to fail in the nominal webs. And, based on the test beam without a concrete web, these webs were taking only half the shear. In a concrete beam in practice this would not be the case, for the concrete would be taking all of the shear. This would lower the point of incipient failure to something like 120 lb. In

Mr. Godfrey. any event, it is clearly demonstrated that a beam with the shear allowance of this report would be sure to crack when the bare safe load was placed upon it. That is, it would have a shear crack—have the value of the concrete web destroyed as a shear carrying element of the design. And yet that same concrete web is counted on to carry a large portion of the shear. Here is one of the many questions that no one has answered. How can a concrete web carry 40 lb. per sq. in. of shear when, by the very conditions of the design, it will be cracked when the safe load is reached?

The new report gives a hypodermic to the rodded column to prolong its life by reducing the unit stress and stiffening up the requirements for the spacing and size of ties. A shaft of plain concrete with a slender rod in each corner and wires enough to hold those rods in place during the hardening of the concrete was a standard and accepted column only a few years ago. This thing died in 1916 in spite of a whole army of engineering doctors who defended it and endeavored to keep it alive.

The new report makes the rodded column a little less dangerous, and New York engineers are squealing because they can design cheaper by the New York Building Code.

The unit compression of beams and slabs is jumped from 650 to 800 lb., and the bending moment of continuous beams and slabs is reduced from $WL/12$ to $WL/16$. Here is a degradation in design that means a beam or slab by this standard would have to be increased in strength 64 per cent to bring it up to the standard of the 1916 report. In spite of this New York engineers are squealing again because they can design beams and slabs cheaper by their code. The safety of a building code is not to be judged by the size of the city: neither is the safety of the buildings.

In 1915 before this convention a report was made of some very elaborate tests alleged to be on reinforced-concrete columns, but in fact on piers, and so defined in the then existing Joint Committee report, and chunky piers at that. On the basis of these tests on piers (as I pointed out) standards of designing columns were made and defended. The present report says a column is something three times as high as its diameter. What are dictionaries and facts when you have a lot of nice tests that would be worthless without that definition? What is the use of majority votes if they cannot make a column out of any old thing that the majority sees fit to make it of? And what are majority rulings for, if individual engineers cannot take a smug position behind a whole committee and fire off rulings to their hearts' content, with one master ruling to hide behind, that no excuse whatever is to be offered by the committee or any member of it for anything the committee may see fit to put in their report?

By their absurd formula for hooped columns, by the introduction of the hooped column with a cast-iron core, by uniform spacing of bars across a column footing out to the edge, by prohibiting sagging of reinforcing rods, by considering the counterfort of a retaining wall a triangular beam, the committee seems to be feeling out the profession to see how many pet hobbies they can put across; this to say nothing of the utterly impracticable

Mr. Godfrey.

tables on concrete mixtures and rules for slump tests, also hobbies on designing—designing concrete mixes. Some one had a hobby on stopping joints in the middle of a beam or slab with a feather edge (Section 71). This, I understand, has not much support. This petty log rolling would be amusing if it did not play with human lives and valuable property as a pawn.

AMERICAN CONCRETE INSTITUTE.

BUSINESS REPORTS

REPORT OF THE BOARD OF DIRECTION TO THE CONVENTION.

The Institute has come through a rather bad business year in good shape although the progress has not been as great as we had hoped to make.

On February 1, 1921, we had 542 active members and 85 supporting members, a total of 627. We lost in the year one supporting member and increased our active membership by 180, giving a total membership February 1, 1922, of 806, a gain of 29%. The increase in two years has been 378, or more than 88%.

Since we had a membership of 1000 as a goal on Feb. 13 and not on Feb. 1, it seems only proper to get a little ahead of the formal annual report by announcing that between Jan. 31 and Feb. 13 we added 51 new members, bringing the total to 864, 237 more than at our last convention, and within 136 of the goal of 1000, making the increase since our last convention 37%.

It is reasonable to expect that we may realize our immediate goal of 1000 before the convention adjourns or very soon after as a result of the work done by local membership committees in twenty-two cities and direct from the secretary's office. Cleveland is a fruitful city and if our active members will give us a boost we should be able to adjourn this convention happy in the thought that the Institute has made promising strides. May we have your help? After we have passed the 1000 mark, the growth should be easier.

Business conditions last year dissuaded us from active solicitation of supporting members but there are many corporations in the field who no doubt will come to see how very definite is the service given by the Institute to the progress of concrete construction. As the success of these corporations is wrapped up in that progress, we expect to persuade more and more of them that in helping the Institute to do its work they are really helping themselves in the broadest and soundest way.

On the subject of finance, the Directors established a budget system in the fall of 1919 setting up definite sums for each item of expense and with an eye fixed on the probable yearly income. You will be interested to know that we have kept within our income. For the last fiscal year ending June 30, 1921, we received $14,138.08 and expended $11,955.91.

The 1920 *Proceedings* were paid for out of the money, leaving the 1921 *Proceedings* to be paid out of our surplus and the income for this fiscal year. These *Proceedings*, Vol. 17, cost $5306.78, and included the valuable Westergaard-Slater Report.

The Receipts and Expenditures for the fiscal year, July 1, 1920, to June 30, 1921, were as follows: [These figures are included in an abstract of the auditor's report elsewhere in these papers and are therefore omitted from this report as presented at the convention.—ED.]

The complete audit of the Public Account, Albert E. Horne, to the end of the last fiscal year will appear in the Proceedings (P. 410).

There is great need for more money to undertake construction work for the members, investigation and reports on concrete products, large sized column tests to warrant the adoption of new column formulae, more work to determine the effect of water in the strength of concrete, determination of methods to more closely forecast the strength of concrete, a new series of tests to give us more knowledge of the effect of length of time of mixing on the strength of concrete, etc., etc. All of this work should be definitely mapped out in the interest of the proper growth of the industry.

As a result of the adoption of the report of the Committee on Organization, whose work was freely reported in the News Letters, we have greatly increased and more definitely assigned committee activities. A year ago we had 21 standing committees. Now we have provided for 33—three of them not yet fully organized. One old committee has been discontinued; the present program includes 13 new ones. There are five general committees, seven on engineering, six on special structures, six on contractors' problems, seven on concrete products.

This reorganization and reassignment of committee work has given evidence of a sincere effort to the developments of the field. Appreciation has been shown in the increased membership. A considerable number of new members from the products manufacturing field shows that that field is responsive to an increased consideration of its problems. Concrete products manufacturers of the more progressive kind have been quick to see that the Institute can do for their branch of the industry what no other organization can do in creating standards.

There is also an increased interest in the Institute among contractors. The Board has made a special point of furthering a policy of bringing the Institute's work into closer accord with the practical problems—endeavoring to translate into practice on the work our constantly developing knowledge of how to make good concrete.

The Institute has for several years back leaned strongly to engineering problems, to theory and to research and it is not in carrying out its present policy departing from that line of study, but is trying to reduce it to the terms of its application to the job.

It has also been the policy of the Institute to bring its activities more and more to the attention of those who are and who might be interested by the publication of its occasional News Letters and by active co-operation with the technical press which must always be an ally of the technical society in presenting its message. The Institute has recently published a little pamphlet briefly setting forth what it is doing and what it is trying to do and has given it considerable circulation with good results in getting new members. A new and slightly revised edition of this pamphlet will soon be published. There will also be issued as soon as practicable after this convention a News Letter presenting a digest of its work with the idea not only of reporting the convention in skeleton form to those who could not attend, but also to put into the hands of members ammunition with which to seek new membership support. The Institute does not wish to dwell unduly on the subject of membership but every additional member adds to our moral strength in upholding our standards and reduces the per capita cost of these publications, leaving a greater margin for increased activity.

Most of the committees are functioning, although not all have reports this year. It will probably be impossible always to consider reports from all of our committees at any one convention. Organization in many cases was late in being completed, but some of the committees have accomplished a great deal in a short time. Committee work of this kind must of course be done at some sacrifice of immediate personal consideration. Those who are very active in committee work, however, see the broader results which are possible and the things which may be achieved for the industry as a whole from which they will have their proper share of benefit. It is inevitable also that some members of the Institute should feel that they should have been appointed to certain committees, but appointment depends upon many considerations, and it is not practicable nor advisable to have a committee made up of too many men of a single line of thought. Every committee man was appointed for a definite reason, in order to give to each committee a breadth of thought and many angles of thought on each committee problem. If the chairman and secretary of a committee get to work too late in formulating a report, that report cannot have in it the breadth and soundness which can be given it by the combined thought of all the members of the committee. Next year's committee work should be started early. If members are geographically scattered, the work takes a much longer time. The Board of Direction urges very strongly that each committee's program for next year be gotten started promptly so that whatever report is made, it will be as broad and as mature in its observations and specifications as a broadly experienced personnel will permit. The chairman and secretary of each committee must of necessity initiate a great deal, but it should not stop there.

It is not necessary to comment on the work of this year's committees. That work is largely here before you at this convention. A great deal of committee work has been done carefully and thoroughly, and the reports presented are well considered, and mark real advance in the progress of our field. The program seems to be stronger than usual in subject matter which attempts to solve the problems that are really bothering the men who are doing work of one kind or another in the concrete field. That is what the Institute is for. We'd like to have you feel that and carry the word to others who are not with us now.

In conclusion the Board wishes to repeat that in the last year, a period of unusual business inactivity, the Institute has increased its membership 37 percent; has borne an increased cost for its publications and yet is in a little better financial condition than a year ago and has produced a better convention program.

ABSTRACT OF MINUTES OF MEETINGS OF THE BOARD OF DIRECTION.

MEETING, NEW YORK CITY, MAY 6, 1921.

Present: President H. C. Turner, Messrs. Wason, Pearson, Boyer, Secretary and Treasurer Whipple.

The secretary reported that in accordance with action of the Board Feb. 16, A. E. Horne made an audit of the Institute accounts for the period from June 30, 1920, to February 15, 1921, inclusive, and that certified copy of the audit was furnished the retiring treasurer, R. W. Lesley, at satisfactory relinquishment of responsibility.

The treasurer reported an Institute account opened at the National Bank of Commerce, Detroit, and that the Institute's $3000 in Victory Bonds had been transferred for safekeeping from the Girard Trust Co., Philadelphia, to the National Bank of Commerce, Detroit.

President Turner reported that in accordance with action of the Board of Direction February 16 he obtained a bond for the secretary and treasurer in the sum of $10,000.

The treasurer presented a survey of the Institute's finances for the fiscal year ending June 30, 1921, showing receipts and expenditures for the first ten months of the fiscal year and an estimate of receipts and expenditures for the remaining two months, as compared with the financial budget made up Oct. 1, 1920. This indicated a total of $14,649.12 income for the fiscal year, expenses for the same period of $12,028.23, giving, according to the estimate, a surplus of $2,620.79.

The secretary was authorized to investigate facilities for and general advisability of holding the next convention of the Institute at Cleveland, Pittsburgh, or Atlantic City. The Secretary presented a report of a special Committee on Organization (Subsequently reconstituted with the same membership as the Advisory Committee, G-2), appointed in February to review the Institute's technical activities and to make recommendations as to the committee reorganization personnel, and subject matter of investigation. The report was received and its recommendations with slight amendment adopted. The president was authorized to express to the members of the Committee on Organization appreciation of their detailed work in connection with the report. Because this report involves not only the immediate program of various committees, but the appointment of new standing committees and the adoption of some matters in relation to general policy, the following outline of these more prominent features of the report is presented herewith:

In setting down an outline of the Institute's general policy, the report of the Committee on Organization reminds the Institute membership of the breadth of field of work of the Institute set forth under the heading "Objects" in the original charter:

"To disseminate information and experience upon and to promote the best methods to be employed in the uses of cement by means of Convention, the reading and discussion of papers upon materials of a cement nature and their uses . . . and to circulate among its members by means of publications, the information thus obtained."

In its relations to the work of other organizations, it is maintained that the Institute should adhere strictly to the right of independent action upon any matter within its field, but to insure at all times a knowledge of progress in other organizations, there shall be officially designated visiting memberships from our committees in similar committees of other organizations whenever this is agreeable to the other organizations affected, and this arrangement be made reciprocal by welcoming visiting memberships from other organizations.

In considering the membership of the Institute as divided among three groups of interest (Engineering Design and Inspection, Contracting and Concrete Products Manufacture), it seemed desirable to consider this grouping merely as classifying the viewpoint and scope of interest of the members of the Institute and not to use it as a basis for dividing Institute work, as so many of the problems of the field depend for their solution upon the co-operation of two or more of the groups. On this same basis, it was recommended that the Convention of the Institute should not be held in overlapping sections. As affecting work of committees, certain committee rules and policies were adopted. These are summarized as follows:

The reports of all committees shall be made to the Secretary's office, which shall be the clearing house for all of the work of the organization.

Each Institute committee shall have a secretary whose duties will be to co-operate with the chairman of his committee and to the fullest by arranging meetings and by carrying on correspondence.

The chairman and secretary of each committee shall advise the Secretary of the Institute of the program of the work and suggest for consideration of the Advisory Committee changes in committee personnel.

The work of committees is to be assigned definitely by the Board of Direction.

The sub-division of committee work should be, whenever possible, in subcommittees arranged by each committee for itself, subject to the approval of the Board of Direction through the Advisory Committee.

Whenever the work of one Institute Committee is of special concern in the work of another Institute Committee, it is recommended as a general policy that there be an exchange of official representation between committees. Any committee may ask for this representation at any time. The member appointed to represent his committee in the deliberation of another committee to which he is accredited, shall be definitely instructed by his committee, so as to act for his committee rather than as an individual.

In organizing committees the President and Board of Direction after deciding upon the chairman and secretary, shall defer as far as possible to their ideas in the appointment of additional committee members.

As a general policy the Institute shall do as much as possible of its work through independent committee action and with the aid of visiting members on the committees of other societies rather than through Joint Committees, it being felt that a report of a Joint Committee is apt to represent a compromise which does not adequately set forth the ideas of each parent organization.

Institute committees are now, under the Board's action on the Organization Committee's report, in six classifications:

1. General Committees
2. Joint Committees
3. Committees on Engineering Design and Inspection
4. Committees on Special Structures
5. Committees on Construction
6. Committees on Concrete Products Manufacture.

Under the heading General Committees, we have the following:

GENERAL COMMITTEES.

G-1 Committees, consisting of the chairmen of all other committees. It will meet at least once a year at the convention when the program for the ensuing year will be considered especially with a view to co-ordinating the work of technical committees, avoiding duplications and for the purpose of recommending what major subjects should be emphasized in the year's work to make the program of the most immediate practical value. Between meetings of the committee on Committees, this work will be carried forward by a small Advisory Committee.

G-2 Advisory. This committee is to consider and recommend to the Board of Direction the acceptance or rejection of reports proposed for presentation at the convention with a view to limiting the consideration of some subjects and increasing the consideration of others, to make the best balanced convention program; to assume duties which were formerly assigned by the Board of Direction to the Program Committee and take over the duties of the former committee on Publication. Its work will be to aid the Board of Direction in the Institute's technical activities through committees, in the preliminary details of convention and the preliminary work of issuing the annual Proceedings.

Other General Committees are G-3 Form of Standards, G-4 Nomenclature, G-5 Wason Medal.

JOINT COMMITTEES.

Committee J-1, the Institute's representation on the Joint Committee, is acting with representatives of the American Society for Testing Materials, The American Society of Civil Engineers, The Portland Cement Association, and the American Railway Engineering Association.

J-2, Concrete Pipe, is the Institute's representation on the Joint Committee on Concrete Pipe on which are represented the American Society of Civil Engineers, American Railway Engineering Association, United States Bureau of Public Roads, American Society for Testing Materials and American Association of State Highway Officials.

ENGINEERING DESIGN AND INSPECTION.

E-1.—Reinforced Concrete Building Design and Specifications (Formerly Reinforced Concrete and Building Laws).

E-2.—Standardization of Units of Design.

E-3.—Research.

E-4.—Fire Proofing.

E-5.—Aggregates.

E-6.—Destructive Agents and Protective Treatments (A new committee).

E-7.—Waterproofing (A new committee).

SPECIAL STRUCTURES.

S-1.—Reinforced Concrete Chimneys.

S-2.—Reinforced Highway Bridges and Culverts.

S-3.—Reinforced and Plain Concrete Sewers and Conduits. (Eliminating from the consideration of this committee everything of the pre-cast type of construction.)

S-4.—Concrete Storage Tanks. (Cast in place structures only.)

S-5.—Reinforced Concrete Houses. (Cast in Place Structures only.)

S-6.—Concrete Roads and Pavements.

CONTRACTING.

C-1.—Contractors' Plant.
C-2.—Concrete Floor Finish.
C-3.—Treatment of Concrete Surfaces.
C-4.—Metal Forms.
C-5.—Central Mixing and Central Proportioning Plant. (Such a committee was not appointed last year.)
C-6.—Field Methods.

CONCRETE PRODUCTS MANUFACTURE.

(All these committees are new, replacing a former committee on Concrete Products.)

P-1.—Standard Building Units (Including block, brick, and structural tile).
P-2.—Manufactured Dimension Stone. (This committee was not organized last year.)
P-3.—Building Regulations for the Use of Concrete Units. (This committee was not organized last year.)
P-4.—Concrete Staves.
P-5.—Fire Resistance of Concrete Building Units.
P-6.—Products Plant Operation.
P-7.—Pipe, Drain Tile, and Conduit.

The Report of the Committee on Organization recognized the need in the near future of other committees in the Products Field whose work should include such matters as Tanks, Trays, and Burial Vaults; Roofing Units; Posts, Poles and Lighting Standards, and other concrete products specialties and ornamental work.

MEETING, NEW YORK CITY, OCT. 6, 1921.

Present: President Turner, Past Presidents Humphrey and Hatt, Messrs. Boyer, Ashton, Abrams, Pearson, and the Secretary.

Minutes of the Meeting of the Board of Direction held May 6 having been submitted by mail to the members of the Board and approved by a majority of the members of the Board between May 24 and June 6 were not read.

Frank C. Wight, editor of Proceedings, Volume 17, submitted a report of the cost of the Proceedings totalling $4,993.79, not including postage and mailing, nor the compensation of the editor.

The secretary was requested to advise contributors, that papers will be considered for presentation with the understanding that the Advisory Committee shall have the authority to exercise its discretion in editing material before final presentation in the Proceedings.

Ballots on the election of members of a Nominating Committee were canvassed with the result that the five having the highest number of votes were as follows: S. C. Hollister, R. F. Havlik, N. M. Loney, J. H. Libberton, J. H. Chubb.

The report of the auditor, A. E. Horne, for the fiscal year ending June 30, 1921, which had been previously submitted to the Board was considered on the suggestion of President Turner on the showing of a surplus of $9,407.15. While these figures were admittedly correct on the basis of the Institute's system of bookkeeping, it is pointed out as offering the possibility of misunderstanding the Institute's financial condition, and that in publishing the auditor's report there should be appended an explanatory note to the effect that expenses to which the Institute was committed at the time this report was made (which commitments do not appear on the books of the Institute, chief among them being the cost of the Proceedings) constitute a considerable item which stands as a legitimate offset amounting to about $5,000 to the apparent surplus.

The report of the auditor was accepted, the secretary authorized to file it and publish a digest of it in the next annual Proceedings.

Special note was taken of the death of W. C. Robinson, vice president and chief engineer of the Underwriters' Laboratories, Chicago, a member of the Institute for years, held in high personal esteem and valued for his wise council especially in matters of fireproofing.

There was a discussion of the likelihood of a successful membership campaign in the months prior to the next convention. It was the consensus of opinion of the Board that the secretary should further defer active membership work until November or December.

The secretary was requested to prepare a budget covering the estimated income and expenditures for the remainder of the fiscal year and submit it to the Finance Committee.

The secretary presented an analysis of geographical distribution of the Institute members, more particularly in respect to the previous action of the Board that Pittsburgh, Cleveland, or Atlantic City be considered as a convention place for 1922. It was found that 79% of the total membership of the Institute and 90% of the membership of the Institute within the United States lies within 500 mile circles drawn around the cities of Chicago, Pittsburgh, Cleveland and Atlantic City.

The secretary was authorized to make arrangements for the convention to be held in either Pittsburgh or Cleveland, with a preference for the latter, on Monday, Tuesday, Wednesday and Thursday, February 13, 14, 15, and 16, 1922.

It was brought to the attention of the Board that a request had been received from Secretary Hoover that the Institute appoint a representative from its membership to co-operate with the Building Code Committee of the Department of Commerce. The Advisory Committee had recommended that A. R. Lord, chairman of the Institute Committee E-1 on Reinforced Concrete Building Design and Specification, be appointed to co-operate with the Building Code Committee. The Board approved this recommendation and the secretary was requested to notify Mr. Lord and the Department of Commerce.

Detailed consideration was given to the proper elements for the next convention program.

The following resolution was adopted:

Whereas, American Standard Specifications for Portland Cement No. 1 have been approved by the American Engineering Standard Committee representing the United States Government and the engineering and technical societies of this country, and

Whereas these specifications are the result of many years of investigation and study by co-operating committees consisting of the leading experts in this country and representing the United States Government, the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering Association, the American Institute of Architects, and other engineering organizations, and

Whereas the rapid extension in the uses of portland cement concrete in important structures in which the use of materials of uniform quality is highly essential for safe construction,

Therefore, Be It Resolved that the Board of Direction of the American Concrete Institute requests its members to adopt the American Standard Specifications for portland cement and urges them to co-operate in securing the adoption of these standard specifications by all engineers and architects engaged in portland cement concrete construction.

NOTE: These specifications are designated as "American Engineering Standard Specifications No. 1" and may be obtained from The American Society for Testing Materials, 1315 Spruce Street, Philadelphia, Pa.

MEETING AT CLEVELAND, FEB. 13, 1922, 11.00 P. M.

Present: President Turner, Vice President Anderson, Past President Humphrey, Messrs. Boyer, Ashton, Pearson, Abrams, and Secretary and Treasurer Whipple.

The secretary read and revised a report of the Institute's condition and its activities, which was approved for presentation by the secretary to the convention at the business session February 14.

MEETING AT CLEVELAND, FEB. 14, 1922, 12.30 P. M.

Present: President Turner, Vice-President Anderson, Past President Humphrey, Messrs. Boyer, Ashton, Pearson, Abrams, and Secretary and Treasurer Whipple.

Meeting was devoted to a general discussion of the activities of the Institute and no definite action was taken.

MEETING AT CLEVELAND, FEB. 15, 1922, 12.30 P. M.

Present: President William P. Anderson, Vice-President A. E. Lindau, Past Presidents Humphrey, Wason and Turner, Messrs. Ashton, Pearson, Abrams, Nichols, and Secretary and Treasurer Whipple.

Harvey Whipple was reappointed Secretary.

Authorization was given that all voucher checks on funds of the Institute be executed with the signatures of the three officers as follows: President W. P. Anderson, (or in his absence or incapacity Vice President A. E. Lindau) and Treasurer Harvey Whipple, and Secretary Harvey Whipple; the action to be certified to the National Bank of Commerce, Detroit, President Anderson was requested to renew at its expiration the bond of the secretary and treasurer in the sum of $10,000, with the American Surety Company, New York.

Retiring President Turner reported a contract entered into with the Winston Company for printing the 1922 Proceedings, this contract being on record in the secretary's Office.

Frank C. Wight was reappointed editor of the 1922 Proceedings.

The secretary was requested to take up with the membership committees in various major cities the matter of increasing the membership to 1000 by the end of the present fiscal year and urge efforts to that end.

The Secretary was requested to express for the Board of Direction its special appreciation of the service of Mr. Frank Hitchcock, Chairman of the Washington, D. C., membership committee, it being the first to go "over the top" in securing more than its quota of new members.

Messrs. Lindau, Abrams, and Nichols were appointed to constitute the Finance Committee of the Board.

Messrs. Turner, Boyer, and Lindau were appointed to serve with the President and the Secretary as the Executive Committee.

The Advisory Committee G-2 was continued with the same membership as last year, as follows: S. C. Hollister, chairman, A. T. Goldbeck, E. J. Moore, N. M. Loney, R. F. Havlik, Harvey Whipple, secretary.

The secretary was authorized to refer the existing standards of the Institute to the Advisory Committee on the report of Committee G-3, Form of Standards that none of them conforms to the new form of standards recently adopted by the Board, with a view to their being referred back to their originating committees for revision in form and in substance as may be required to bring them up to date, the Standards as revised to be reported back to the Institute for early adoption and publication to make them available to the engineering profession.

The Advisory Committee was authorized to fix the date January 1, 1923, as a time limit for the announcement of and presentation in outline form of reports and papers which are to be included in the program of the 1923 convention.

The Secretary was requested to ask the members of the Institute what in the program of the Cleveland convention they liked best with a view to being guided somewhat by their advice in the work of the Advisory Committee in outlining the 1923 program.

The sum of five hundred dollars ($500) was appropriated as the nucleus of a fund of about $6000 to be known as the "Concrete Building Units Fire Test Fund" and the Institute's Committee P-5: Fire Resistance of Concrete Building Units authorized to solicit the additional funds necessary to pay for the tests as outlined by Committee P-5 in its report, none of the Institute's appropriation to be made available until all of the funds necessary for the tests are fully subscribed.

MEETING AT CLEVELAND, FEB. 15, 10.30 P. M.

Present: President William P. Anderson, Vice President Lindau, Past-Presidents Humphrey, Wason, and Turner, Messrs. Boyer, Ashton, Pearson, Abrams, Nichols, and Secretary and Treasurer Whipple.

The secretary was authorized upon receipt of request from the Secretary of the Joint Committee on Specifications for Concrete and Reinforced Concrete for additional funds, to meet the request with a payment pro rata with other parent organizations not in excess of $100.

The Institute's representatives on the Joint Committee on Specifications for Concrete and Reinforced Concrete were instructed to urge strongly upon the Joint Committee the desirability of a comprehensive series of field tests to determine the practicability of the recommendations contained in Section 28 of the Joint Committee Progress Report, the Institute pledging itself to the extent of its ability to co-operate in the plan and execution of such tests.

AUDITORS REPORT.

ALBERT E. HORNE,
301 Hague Ave.

DETROIT, July 14, 1921.

Mr. Harvey Whipple, Sec'y. and Treas.
American Concrete Institute,
New Telegraph Building,
Detroit, Michigan.

Dear Sir:

In accordance with your instructions, we have made an examination of the books and records of the American Concrete Institute for the period from June 30, 1920, to June 30, 1921, for the purpose of verifying the Cash transactions of the period and presenting the financial condition of the Institute at that date.

We submit herewith two exhibits as follows: Exhibit A—statement of condition as of June 30, 1921, Exhibit B—statement of receipts and disbursements from June 30, 1920, to June 30, 1921.

The Cash in Bank, amounting to $4,090.81 as shown by the Cash Book, was verified by reconciliation with the statement rendered by your depository as of June 30, 1921. During our examination paid checks, all of which were properly approved by your President or Vice President and Secretary, were seen for all disbursements.

The remaining items in statement of condition are shown in accordance with the records, and have not been further verified by us.

We have not attempted to determine the income which should have been derived during the year, and have restricted our examination to the disposition of all cash shown to have been received.

We hereby certify:

1. That all cash shown to have been received has been accounted for, and that we have seen satisfactory evidence for all disbursements.

2. That the Cash in Bank, amounting to $4,090.81, June 30, 1921. was on deposit on that date; also that the Imprest Cash amounting to $300.00 was in the hands of the Secretary.

3. That the statement of condition (Exhibit A) is in accordance with the records and subject to the foregoing comments, and in our opinion

properly presents the financial condition of the American Concrete Institute at June 30, 1921.

Respectfully submitted,

(Signed) A. E. HORNE.

P. S. The Secretary calls my attention to the fact that bills for the printing of the Proceedings of the Institute, Volume 17, 1921 Convention, have not been presented. These bills are properly a charge against the surplus of June 30, 1921, given in my report.

(Signed) A. E. HORNE.

AMERICAN CONCRETE INSTITUTE.

STATEMENT OF CONDITION.

June 30, 1921.

Exhibit A.

ASSETS.

Cash in Bank	$4,090.81	
Cash Imprest	300.00	
Total Cash		$4,390.81
Victory Bonds		3,000.00
Accts. Receivable—Dues	$932.20	
Accts. Receivable—Misc.	33.25	
		965.45
Inventories:		
405 Proceedings @ $3.75	$1,518.75	
722 Journals @ $0.375	270.75	
Total Inventories		$1,789.50
Total Assets		$10,145.76

LIABILITIES.

Accounts Payable	$738.61	
Surplus	9,407.15	
	$10,145.76	$10,145.76

Analysis of Cash Balance:		
Balance in Bank as per Statement June 30, 1921	$4,090.81	
Statement June 30, 1921		
Imprest Cash	300.00	
		$4,390.81

No checks outstanding
June 30, 1921.

AMERICAN CONCRETE INSTITUTE.

CASH RECEIPTS AND DISBURSEMENTS.

As shown by Cash Book

July 1, 1920, to June 30, 1921.

Receipts.

Balance on hand June 30, 1920	$1,908.64
Dues	5,468.33
Proceedings	1,271.42
Contributing Members	6,263.31
Miscellaneous	1,135.02
Total receipts	$16,046.72

Disbursements.

Foreign Exchange	$1.21	
Salaries	2,851.01	
Office printing and Supplies	1,346.15	
Proceedings	2,814.60	
Travel	525.22	
President's Office	12.70	
Postage	432.37	
Convention	1,179.68	
Misc. Expense	1,694.95	
Imprest Cash	100.00	
Flat Slab Report Expense	998.02	
Total disbursements		$11,955.91
Balance in Bank June 30, 1921		$4,090.81

The receipts and expenditures in the present fiscal year from July 1 to February 1 were as follows:

RECEIPTS.

Dues—Active	$4,103.82	
Dues—Contributing	4,040.02	
Proceedings	383.71	
Misc. preprints sale	59.55	
Interest—Commercial account and V. Bonds	146.09	
Contribution—Far Rockaway Fire Com.	102.80	
		$8,835.99

DISBURSEMENTS.

Salaries	$1,483.33	
Postage	205.36	
Proceedings	4,941.96	
Printing Preprints	292.91	
Travel	193.68	
Office Supplies and misc. ptg.	237.10	
Far Rockaway Fire Committee additional expense	202.80	
News Letters, Membership list and pamphlets	705.94	
Miscellaneous expenses	207.30	
		8.470.38

SUMMARY.

Balance in Bank July 1, 1921—auditor's report	4,090.81
Total receipts	8,835.99
	$12,926.80
Total Disbursements	8,470.38
Balance in bank	$4,456.42
Balance shown by National Bank of Commerce Statement of February 1, 1922	4,456.42*

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*In addition to this we have $3,000 in Victory Bonds.

MEMBERS AMERICAN CONCRETE INSTITUTE.

An Asterik (*) designates a Supporting Member.

Members will please send to the Secretary a correction of any error of name or address in this list.

Abbott, C. J., Cleveland, Ohio. (Hydraulic Steelcraft Co.)

Abe, Mikishi, 24 Mita S Temple, Shiba, Tokyo, Japan.

*Aberthaw Construction Co., 27 School St., Boston, 9, Mass. (L. C. Wason.)

*Aberthaw Construction Co., 27 School St., Boston, 9, Mass. (L. C. Wason.)

Abrams, Duff A., Lewis Institute, 1951 W. Madison St., Chicago, Ill.

Accevedo, Octavio, Palacio di Gobierno, Santo Domingo, Republica Dominicana.

Aeby, Norman O., Perth Amboy, N. J.

Affleck, B. F., 210 S. LaSalle St., Chicago, Ill. (Universal Portland Cmt. Co.)

Aktiebolaget Marinbetong, Nykoping, Sweden. (E. V. Carlsson.)

Albright & Mebus, Land Title Bldg., Philadelphia, Pa. (Charles F. Mebus.)

Aldridge, E. V., 208 S. LaSalle St., Chicago, Ill. (Universal Portland Cmt. Co.)

Alger, H. H., 244 Madison Ave., New York, N. Y. (Turner Construction Co.)

Alin, A. L., 45 Regeringsgatan, Stockholm, Sweden.

Allen, O. T., 208 S. LaSalle St., Chicago, Ill. (American Steel & Wire Co.)

*Allentown Portland Cement Co., Allentown; Pa.

Alliance Hollow Cement Block Co., Northampton, Pa. (J. H. Van Middlesworth.)

Allison, A. C., 90 Main St., Malone, N. Y. (The Allison Concrete Works.)

*Alpha Portland Cement Co., Easton, Pa. (L. Anderson, Jr.)

*Ambursen Construction Co., 186 Fifth Ave., New York City.

American Bureau of Shipping, 66 Beaver St., New York City.

American Can Co., 120 Broadway, New York City. (C. E. Preis.)

American Cement Paint Co., Box 294, Chattanooga, Tenn. (W. P. D. Moross.)

American Cement Tile Mfg. Co., Oliver Bldg., Pittsburgh, Pa.

American System of Reinforcing, 10 S. LaSalle St., Chicago, Ill. (Arthur A. Clement.)

Amsterdam Bldg. Co., Inc., 140 W. 42nd St., New York, N. Y. (T. H. Maxwell.)

Anderson, John Edward, Box 40, Te Puke, Bay of Plenty, New Zealand.

Anderson, Walter W., 422 Newbury St., Boston, Mass.

Arbour, Erwin F., Fifth Ave. and Oak St., Bayshore, Long Island.

Archibald & Holmes, Continental Life Bldg., Toronto, Ont. (A. R. Holmes.)

Arnold Stone, Brick & Tile Co., Inc., Lem Turner and 47th Sts., Jacksonville, Fla. (M. A. Arnold.)

Arquisola, Jose, Poblacion, Romblon, Philippine Islands.

Asbestos Shingle, Slate & Sheathing Co., Ambler, Pa. (Richard V. Mattison.)

Ash Grove Lime & Portland Cement Co., Box 1132, Kansas City, Mo. (A. Lundteigan.)

Ashley, G. F. Care of H. Ashley, 2010 Vallejo St., San Francisco, Cal.

Ashton, Ernest, Allentown, Pa. (Lehigh Portland Cement Co.)

Associated Metal Lath Mfrs., Edison Bldg., Chicago, Ill. (Wharton Clay.)

Associated Portland Cement Mfrs., Ltd., Park House, Gravesend, England. (H. R. Cox.)

Association of Professional Engineers, Winnipeg Public Library, William Ave., Winnipeg, Man.

Ayers, F. C., 250 W. Lafayette Blvd., Detroit, Mich. (Truscon Steel Co.)

Ayres Concrete Products Co., Inc., 3602 Knorr St., Tacony, Philadelphia, Pa. (Joseph L. Ayres.)

Babbitt, Arthur B., 268 N. Water St., Kent, Ohio. (The Kent Machine Co.)

Badder, H. Craighill, 27 Southfield Road, Bedford Park, London W4, England. (The Torbay & Dart Co., Ltd.)

Baffrey, Raymond, 1 Rue Danton, Paris, France. (Raymond-Baffrey-Hennebique.)

Baker, Hugh J., Majestic Bldg., Indianapolis, Ind.

Baker, Philip I., 1004 Lindsay Bldg., Winnipeg, Manitoba. (C. A. P. Turner Co., Minneapolis, Minn.)

Ballinger, Walter F., S. E. Cor. 12th and Chestnut Sts., Philadelphia, Pa. (The Ballinger & Co.)

Ballou, Henry W., 10 Weybossett St., 1035 Grosvenor Bldg., Providence, R. I. (Jenks & Ballou.)

Bangor Cast Stone Products Co., Bangor, Maine. (George Cuozzo.)

Barbour, F. A., 1120 Tremont Bldg., Boston, Mass.

Barnard, Ira B., Box 266, Trinidad, Colo. (The Hydraulic Stone & Bldg. Co.)

Barney, W. Joshua, 110 West 40th St., New York City.

Barney-Ahlers Construction Corp., 110 W. 40th St., New York City.

*Barney-Ahlers Construction Corp., 110 W. 40th St., New York City. (John G. Ahlers.)

Barrows, Frank G., 60 Federal St., Boston, Mass. (National Engineering Co.)

Bartlett, G. S., 208 S. LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)

Bartram, George C., 1003-29 Broadway, New York City. (United Concrete Pipe Co.)

Batschelet, J. C., 159 Second St., N. S., Derry, Pa.

Baum, Co., H. W., Walker Bank Bldg., Salt Lake City, Utah. (H. W. Baum.)

Baylson, J. Jacob, N. W. Cor. Hope and Norris Sts., Philadelphia, Pa. (Security Insert Co.)

Beard, Elmer H., 1 Bank St., Room 308, Washington Bldg., Stamford, Conn.

Beard, Frederick S., 133 E. Grand Blvd., Detroit, Mich.

Beard, Theodore W., 71 Wooster, Shelton, Conn. (Waterbury Manufacturing Co.)

Beattie & Cornell, 33 N. Quarry St., Fall River, Mass. (William H. Beattie.)

*Beattie, Roy H., 10 Purchase St., Fall River, Mass.

Becker, W. C. E., 426 City Hall., St. Louis, Mo. (Chief Engr., Building and Inspection.)

Beggs, George E., Assoc. Prof. Structural Engineering, Princeton, N. J. (Princeton University.)

Bell, James C., 12 Twentieth St., Elmhurst, L. I.

Benson, Newton D., 36 Burrington St., Providence, R. I.

Bent Bros., 1714 Eagle St., Los Angeles, Cal. (Arthur Bent.)

Bent, Ernest F., California Glazed Cement Pipe Co., Security Bldg., Los Angeles, Cal.

Bentham, Cecil, 20 Mount St., Manchester, England. (Henry Sims, Ltd.)

Benton, William, Route 4, Station 36, Mt. Clemens, Mich. (W. S. Peacock Co.)

Berberich, Frank J., Room 773, Merchant National Bank Bldg., Los Angeles, Cal.

Berger, Edward H., Connecticut Ave. and Pierce Mill Road, Washington, D. C. (Bureau of Standards.)

Berger Mfg. Co., Canton, O. (A. H. Bromley, Jr.)

Berke, S. R., 291 Main St., Webster, Mass.

Bernier & Son, N., 411 Walden St., Cambridge, Mass. (Napoleon M. Bernier.)

Berry, B. W., 343 Jefferson, Monte Vista, Colo.

Bertin, Rene L., 95 Madison Ave., New York, N. Y. (White Construction Co., Inc.)

Besosa, J. C., 12 San Mateo St., San Turce, Porto Rico.

*Best Co., H. D., Fifth Ave. at 23rd St., Flatiron Bldg., New York City. (H. D. Best.)

Best, Byron G., 225 Lowell St., Ironwood, Mich. (Oliver Iron Mining Co.)

Beyer & Mahnken, Steneck Bldg., Hoboken, N. J. (A. J. Mahnken.)

Billings, A. W. K., Dominion Bank Bldg., Toronto, Ontario. (Canadian & General Finance Co., Ltd.)

BILODEAU, LTEE. IGN., 144 Avenue Renaud, Quebec, P. Q., (Ign. Bilodeau.)

BINDLEY, H. DUNCOMBE, Office of Engineer of Ways and Works, F. M. S. Railways, Kuala Lumpur, F. M. S.

BINSWANGER, S. J., 5520 S. Park Ave., Chicago, Ill.

BIRCHARD CONSTRUCTION CO., 704 Terminal Bldg., Lincoln, Neb. (B. Sampson.)

BIRMINGHAM SLAG CO., 1607-16 Jeff Co. Bk. Bldg., Birmingham, Ala.

*BLAW-KNOX CO., Box 915, Pittsburgh, Pa. (C. D. McArthur.)

BLISS & LAVALLE, 274 Main St., Springfield, Mass.

BLISS WATERPROOF CONCRETE BLOCKS, 34 Union Hall St., Jamaica, N. Y. (John J. Bliss.)

BODYCOMB, WALTER C., 37 Wall St., New York City.

BOLTON PRATT CONST. CO., 112 Prospect Ave., 801 Columbia Bldg., Cleveland, Ohio. (H. N. Cain.)

BOUND BROOK CRUSHED STONE CO., 9 Clinton St., Newark, N. J. (L. Upton.)

BOURNE-FULLER COMPANY, THE, 1600 Hanna Bldg., Cleveland, Ohio. (G. C. Smith.)

BOYER, E. D., 25 Broadway, New York City. (Atlas Portland Cement Co.)

BRASSERT, WALTER, 2006 Race St., Kalamazoo, Mich. (Michigan Silo Co.)

BRAUN, WALTER, 233 S. High St., Columbus, Ohio.

BREED, H. ELTINGE, 507 Fifth Ave., New York City.

BRENNAN, FRANK X., 1409 Ritner St., Philadelphia, Pa.

BROMLEY, ALBERT H., 1131 Cleveland Ave., N. W., Canton, O. (Berger Mfg. Co.)

BROOKLYN CROZITE BRICK CORP., 140 Cedar St., New York City. (J. L. Miner.)

BROOKS, GALE M., 1330 Corn Exchange Bank Bldg., Chicago, Ill. (Atlas Portland Cement Co.)

BROOKS, R. E., CO., 50 Church St., New York City. (Frank I. Ginsberg.)

BROWN, HAROLD P., 120 Liberty St., New York City.

BROWN, H. WHITTEMORE, 248 Boylston St., Boston, Mass. (The Housing Corp.)

BROWN, PHILIP B., 410 London Bldg., Vancouver, B. C.

BRUNER, PRESTON M., 524 Frisco Bldg., St. Louis, Mo. (P. M. Bruner Granitoid Co.)

BRUNNIER, H. J., 312 Sharon Bldg., San Francisco, Cal

BRYANT, HENRY F., 334 Washington St., Brookline, Mass.

BUFFALO SLAG CO., 1070 Ellicott Sq., Buffalo, N. Y. (H. N. Snyder.)

*BUFFALO STEEL CO., Tonawanda, N. J. (J. Joseph.)

BUILDERS MATERIAL SUPPLY CO., 501 Fifth Ave., New York City. (W. M. Rynerson.)

BURNS, HOMER S., Freeport, Texas. (Freeport Sulphur Co.)

BURR, HENRY A., 205 Fairfax Ave., Nashville, Tenn.

BURROUGHS, H. ROBINS, 469 Fifth Ave., New York City.

BURTON, THOMAS E., 1117 Woodrow, Wichita, Kans.

BUSH, A. L., 5313 Angora Terrace, Philadelphia, Pa.
BUTLER BROS. BLDG. CO., Hamm Bldg., St. Paul, Minn. (Walter Butler.)
CAL CHEMICAL CO., 28 N. Jonathan St., Hagerstown, Md. (B. C. Lightner.)
*CALUMET STEEL CO., 208 S. LaSalle St., Chicago, Ill. (A. S. Hook.)
*CANADA CEMENT CO., LTD., Montreal, Que., Canada. (H. S. Van Scoyoc.)
CAPE & CO., LTD., E. G. M., 920 New Birks Bldg., Montreal, Que., Canada.
CAREY CONCRETE CO., 348 Grand Ave., Wisconsin Rapids, Wis. (W. H. Carey.)
CAREY, PHILIP, CO., Lockland, Cincinnati, Ohio.
CARLSON CO., ERNEST F., 244 Main St., Springfield, Mass. (Ernest F. Carlson.)
CARMICHAEL, CHARLES, Harrigan P. O., Dist. Ahmednagar, India.
CASE SCHOOL OF APPLIED SCIENCE, Cleveland, Ohio. (F. H. Neff.)
CASPER RANGER CONSTRUCTION CO., 20 Bond St., Holyoke, Mass. (John P. Bathelt.)
CAST STONE CO., 403-4 Commerce Bldg., St. Paul, Minn. (Harry G. Krum.)
CATLETT, RICHARD H., 27 Haymarket Sq., Boston, Mass. (Richard H. Catlett & Co.)
CAYE CONSTRUCTION CO., INC., 356 Fulton St., Brooklyn, N. Y. (Webster J. Caye.)
CEMENT-GUN CO., INC., Allentown, Pa.
CEMENT-GUN CONST. CO., 900 So. Michigan Ave., Chicago, Ill. (John V. Schaefer.)
CEMENT PRODUCTS CO., THE, 207 W. Fourth St., Davenport, Ia. (Austin Crabbs.)
CEMENTS PRODUCTS CO., Box 123, Wilmington, N. C. (George E. Kidder.)
CHACE, E. M., Room 24, Title Guarantee Bldg., Court and Main Sts., Cincinnati, Ohio.
*CHAIN BELT COMPANY, 444 Wyoming Pl., Milwaukee, Wis. (C. F. Messinger.)
*CHAIN BELT CO., 178 24th St., Milwaukee, Wis. (B. F. DeVine.)
CHAPMAN, CLOYD M., 171 Madison Ave., New York City.
CHAPMAN, HOWARD, 315 Fifth Ave., New York City.
CHERVET, ANTONIN, P. O. Box 275, Beyrouth, Syria.
CHUBB, JOSEPH, 210 S. LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)
CIAMPA, FELIX A., Care of De Santis, Park Ave., Crestwood, N. Y. (Ciampa & Della-Fazia.)
CISSEL, JAMES HARLAN, 1213 S. State St., Ann Arbor, Mich. (University of Michigan.)
CLARK, ROBERT J., Hinckley, Ill.
CLARK, THOMAS S., 304 Baynes St., Buffalo, N. Y.

Clark Cement & Concrete Const. Co., Suite 2, Balford Bldg., 634 So. Western Ave., Los Angeles, Cal. (H. K. Koebig.)
Cement, F. H., & Co., Land Title Bldg., Philadelphia, Pa.
Cleveland Builders Supply & Brick Co., 1029 Leader-News Bldg., Cleveland, O. (E. C. Roberts.)
*Clinchfield Portland Cement Corporation, Kingsport, Tenn.
Cobb, Louis R., 310 N. Fullerton Ave., Montclair, N. J.
Coe, Thomas H., Box 806, Providence, R. I. (O. D. Purington & Co.)
Cohen, A. B., 1 Madison Ave., New York City.
Collins, M. W., 141 Mendron St., Scranton, Pa. (Maloney Oil & Mfg. Co.)
Colonial Concrete Products Co., 518 Fourth Ave., Pittsburgh, Pa. (Sidney F. Galvin.)
"Concrete," 1807 E. Grand Blvd., Detroit, Mich. (Harvey Whipple, Editor.)
*Concrete Mixer Association, 111 W. Washington St., Chicago, Ill. (O. A. Matthews.)
Concrete Products Co., 211 Broad St., Bank Bldg., Trenton, N. J. (I. J. Keuper.)
Concrete Products Co., Finance Bldg., Kansas City, Mo. (E. C. Marqua.)
Concrete Products Co., Winter Garden, Fla. (J. R. Sewell.)
Concrete Products Co. of America, 805 Diamond Bank Bldg., Pittsburgh, Pa. (C. F. Buente.)
Concrete Reinforcing & Engineering Co., 2735 Prospect Ave., Cleveland, Ohio. (G. G. Greulich.)
*Concrete Steel Co., 42 Broadway, New York City. (John F. Havemeyer.)
*Concrete Steel Co., 42 Broadway, New York City. (Walter S. Edge.)
Concrete Steel Bridge Co., 606-626 E. Pike St., Clarksburgh, W. Va. (Frank E. McEnteer.)
Concrete Steel Fireproofing Co., 608 Lincoln Bldg., 333 State St., Detroit, Mich.
Condron Co., 1433 Monadnock Block, Chicago, Ill. (T. L. Condron.)
*Consolidated Expanded Metal Cos., Pittsburgh District, Braddock, Pa. (A. V. Spinosa.)
Consolidated Expanded Metal Cos., Braddock, Pa.
Conwell, E. L., 2024 Arch St., Philadelphia, Pa.
Cooper, W. R., 231 S. Broad St., Philadelphia, Pa. (Wark Co.)
*Coplay Cement Mfg. Co., Widener Bldg., Philadelphia, Pa. (J. F. Twamley.)
Copperstone Products Co., The, 2220 Albion St., Toledo, Ohio. (A. A. Bennett.)
Corben, H. J., City Hall, Darling St., Cape Town, Cape Province, South Africa.
Corridon, Joseph B., 27 West North Ave., Baltimore, Md.
*Corrugated Bar Co., Mutual Life Bldg., Buffalo, N. Y. (A. L. Johnson.)
*Corrugated Bar Co., Mutual Life Bldg., Buffalo, N. Y. (W. S. Thomson.)

*Corrugated Bar Co., 402 Mutual Life Bldg., Buffalo, N. Y. (A. P. Clark.)

Cory, Russell G., 50 Church St., New York, N. Y.

Cowell, Henry, Lime & Cmt. Co., 2 Market St., San Francisco, Cal.

*Cramp & Co., 801 Denckla Bldg., Philadelphia, Pa. (D. L. Kneedler.)

Cranford Construction Co., 407 Gerke Bldg., Cincinnati, O. (John Stewart.)

Crary, Alexander, 1956 Bogart Ave., Borough of Bronx, New York City. (Thompson & Binger, Inc.)

Crisp, Melbourne, 1970 38th Ave., Oakland, Cal. (San Francisco Ship Bldg. Co.)

Cromer & Thornton, Inc., Pratt St. and Ga. R. R., Atlanta, Ga. (Chas F. Cromer.)

Crookes, S. I., 30 Richards-Upton Bldg., Customs St. E., Auckland, New Zealand.

Cunningham & Sons Construction Co., C. S., 80 Boylston St., Boston, Mass. (Daniel Cunningham.)

Curry Concrete Co., Biltmore, N. C.

Danaher, W. E., 481 Delaware Ave., Buffalo, N. Y. (Turner Construction Co.)

Darling, E. H., 47 Home Bank Bldg., Hamilton, Ont.

Davis, B. H., 17 Battery Place, New York City. (Bureau of Standards.)

Davis, E. E., Durant Bldg., Detroit, Mich.

Davis-Heller-Pearce Co., Delta Bldg., Stockton, Cal. (H. Y. Davis.)

Davis, Herbert A., Brooklyn Crozite Brick Co., East New York and Utica Aves., Brooklyn, N. Y.

Davis, Watson, 1418 Rhode Island Ave. N. W., Washington, D. C.

Daw, E. S. H., Somerset House, 9 Martin Place, Sydney, Australia. (Expanded Steel & Concrete Products Co.)

Dawes, Edwin Allen, Williams Bldg., Thomasville, Ga. (Standard Sales Co.)

Dawson, F. M., 3 The Circle, Ithaca, N. Y.

DeFrain Sand Co., 804 Finance Bldg., Philadelphia, Pa. (J. L. Durnell.)

Deinboll, F. K., 1263 Brookley Ave., Cleveland, O.

DeJongh, Juan I., Guatemala City, Guatemala, C. A. (Consul of Holland.)

DeLeuw, C. E., 111 W. Washington St., Chicago, Ill. (Kelker, Gates, & DeLeuw.)

DeLong, Guy W., 22 W. Monroe St., Chicago, Ill. (Truscon Steel Co.)

Delos Engineering Construction Co., 212 Bluff St., Prairie du Chien, Wis. (G. J. Sengst.)

DeMann, W. H., 285-399 Main St., Oneonta, N. Y.

Densmore & LeClear, 88 Broad St., Boston, Mass. (Henry C. Robins.)

Denton & Co., 7 East 42nd St., New York City. (P. E. Eisenmenger.)

Detroit Edison Co., Detroit, Mich. (A. S. Douglas, Chief Engineer.)

Dewey Portland Cement Co., 409 Scarritt Bldg., Kansas City, Mo. (L. Williamson.)

Dexter Portland Cement Co., Nazareth, Pa. (J. B. Brobston.)

Deyo & Co., M. R., Box 431, Peoria, Ill. (M. R. Deyo.)

Di Stasio, Joseph, 29 Broadway, New York City. (Fougner Concrete Steel Co.)

Diver, M. L., San Antonio, Tex.

Dixie Portland Cement Co., Chattanooga, Tenn.

Dixon, DeForrest H., 204 W. 86th St., New York City, N. Y.

Dodson Cement Products Co., The, P. O. Box 947, Wichita, Kansas.

Dole, Ira S., 1328 Tenth Ave., Lewiston, Idaho.

Doleman, Herman F., 507 North Charles St., Baltimore, Md.

Donaldson, W. E., Frick Annex, Pittsburgh, Pa. (Carnegie Steel Co.)

Donohue, J5rry, 608 N. Eighth St., Sheboygan, Wis.

Dowdell, C. O., 6558 Normal Blvd., Chicago, Ill. (National Lime Association.)

Drehman Paving Co., 2622 Parrish St., Philadelphia, Pa. (C. E. Drehman.)

Drew Co., Inc., Fred, Woodward Bldg., Washington, D. C. (Fred Drew.)

Dreyer, Walter, 445 Sutter St., San Francisco, Cal. (Pacific Gas & Electric Co.)

Drinkwater, Ernest, City Hall, St. Lambert, P. Q.

Dunn, Oswald T., 1508 E. 65th St., Chicago, Ill.

Dunn Manufacturing Co., W. E., Holland, Mich. (H. A. LaRoy.)

Dunn Road Machinery Co., Conneaut, O.

Dupuy, Alberto, Apartado 893, Bogota, Colombia, S. A.

Duquesne Slag Products Co., Diamond Bank Bldg., Pittsburgh, Pa. (C. L. McKenzie.)

Dutton, C. B., Monadnock Bldg., Chicago, Ill. (Besser Sales Co.)

Dwyer, John R., Bureau of Standards, Washington, D. C.

Earley, John J., 2131 G. St., N. W., Washington, D. C.

East Texas Cement Works, Jasper, Texas. (J. A. Shaw.)

Eclache, F., 14 Rue Stamboul, Alexandria, Egypt.

Edge, W. S., 149 Broadway, New York City.

Eggleston, Howard, 808 Perdido, New Orleans, La.

Ehlert, E. H., 9119 Harvard Ave., Cleveland, Ohio.

Ekholm, S. L., 896-902 Farrar St., Cadillac, Mich.

Eldridge, H. W., New City, Rockland County, N. Y. (Cement-Gun Co., Inc.)

Elk River Concrete Products Co., Elk River, Minn. (D. W. Longfellow.)

Emley, Warren E., 3705 Keokuk St., Washington, D. C. (Bureau of Standards.)

Engineering News-Record, Tenth Ave. at 36th St., New York City. (Frank C. Wight.)

England, John, Commerce Bldg., Ash St., Sydney, Australia.

Epple, Edward C., Room 1001 Essex Bldg., Newark, N. J. (American Concrete Stone Co.)

Ernst, William, North Greenwood St., Lebanon, Tenn.

Euphrat-Hanley Estimating & Engineering Bureau, 305 Walnut St., Cincinnati, Ohio. (Hunter W. Hanley.)

Evans, Frank M., 1170 Broadway, New York City. (C. H. & R. C. Peckworth, Inc.)

Expanded Metal Engr. Co., 8 W. 40th St., New York City. (W. A. Dittman.)

Fairlie, J. W., Co., 61 Steinert Bldg., Providence, R. I. (J. W. Fairlie.)

Fay, Frederick, 15 Beacon St., Boston, Mass. (Fay, Spofford & Thorndike.)

Feldrappe, M. G., 1523 E. 81st St., Cleveland, Ohio. (H. K. Ferguson Co.)

Ferguson, John A., 1012 Portland St., Pittsburgh, Pa.

Ferguson & Co., J. B., Hagerstown, Md. (J. B. Ferguson.)

*Ferguson, Co., John W., United Bank Bldg., Paterson, N. J. (John W. Ferguson.)

Ferguson, Lewis R., 531 Land Title Bldg., Philadelphia, Pa. (Wig, Hollister & Ferguson.)

*Ferro Concrete Construction Co., Richmond and Harriet Sts., Cincinnati, O. (W. P. Anderson.)

Ferro Concrete Construction Co., Cincinnati, Ohio.

Finley, Raymond A., 55 Sunnyside Terrace, East Orange, N. J.

Fireproof Products Co., Inc., 257 East 133rd St., New York City. (Paul R. Clark.)

Fischer, Jr., Andrew, 935 Union Arcade Bldg., Pittsburgh, Pa. (John F. Casey Co.)

Fischer, L. J., 51 Wall St., New York, N. Y. (Thompson-Starrett Co.)

Fischer, Otto F., Norrmalstorg 3, Stockholm, Sweden. (Betongbryan.)

Fisher-DeVore Construction Co., 35 Blymyer Bldg.; 514 Main St., Cincinnati, O. (Frank F. Fisher.)

*Fiske-Carter Construction Co., 11 Foster St., Worcester, Mass. (Burton C. Fiske.)

Fleischmann, Leon, 531 Seventh Ave., New York City. (Fleischmann Construction Co.)

Fletcher, Austin B., Forum Bldg., Sacramento, Cal.

Fletcher, Richard G., 921 15th St., N. W., Washington, D. C. (Fletcher Fireproofing Co.)

Fogg, Ralph J., Lehigh University, Bethlehem, Pa.

*Foote Company, Inc., The, Nunda, N. Y. (F. L. Dake.)

*Foote Company, Inc., The, 1241 So. Michigan Ave., Chicago, Ill. (F. C. Wilcox.)

Ford, Matt, Caldwell, Kansas.

Foster, Jr., Alexander, 6136 Oxford St., Philadelphia, Pa. (William Steele & Sons Co.)

Foster, William B., 1110 Rodney St., Wilmington, Del. (E. I. Dupont de Nemours & Co.)

Fougner, Herman, 29 Broadway, New York City.

*Fougner Concrete Steel Co., Inc., 29 Broadway, New York City. (H. Fougner.)

Francis, George W., State Highway Depet., Dover, Del.

Francisco, F. LeRoy, 511 Fifth Ave., New York City. (Francisco & Jacobus.)

Frank, Jacob, 9 Monmouth St., Newark, N. J.

*Franklin Steel Works, Franklin, Pa. (E. E. Hughes.)

Fraser, Alexander, Department of Roads, Quebec, Que.

Freeman, John E., 111 W. Washington St., Chicago, Ill. (Portland Cement Association.)

French, A. W., 202 Russell St., Worcester, Mass. (Worcester Polytechnic Institute.)

French, S. H., & Co., 4th and Callowhill Sts., Philadelphia, Pa. (F. T. McBride.)

Friebele, J. F., Broad St. Bank Bldg., Trenton, N. J. (Karns-Smith Co.)

Friel, Francis S., 22 S. Third St., Harrisburg, Pa. (Penn. Dept, of Health.)

Froehling & Robertson, Richmond, Va. (H. C. Froehling.)

Frost & Chamberlain, Slater Bldg., Worcester, Mass.

Fruchtbaum, J., 440 Gurney Bldg., Syracuse, N. Y. (Truscon Steel Co.)

Furber, Pierce P., 638 Hamm Bldg., St. Paul, Minn. (Butler Bros.)

Fusejima, Shinkuro, P. O. Box 572, Champaign, Ill. (Univ. of Illinois.)

Gabriel Steel Co., 1150 Penobscot Bldg., Detroit, Mich. (W. F. Zabriskie.)

Gale, L. E. American Trading Co., Hankow, China.

Garcia, Antonio, Calle de Rodriguez No. 118, Torreon, Coahuilla, Mexico.

Gardiner, J. B. W., 30 Church St., Hudson Terminal Bldg., New York, N. Y. (Gardiner & Lewis.)

Gardner, Franc E., 3123 Bloomingdale Rd., Chicago, Ill. (Gardner-Barada Chem. Co.)

Gawthrop, Alfred H., Wilmington, Del. (American Car & Foundry Co.)

Gedney Construction Co., Hastings, Neb. (Kenneth H. Gedney.)

General Building Co., Inc., 524 Harrison Ave., Boston, Mass. (H. W. Marshall.)

General Fireproofing Co., Youngstown, O. (W. B. Turner.)

*Giant Portland Cement Co., Pennsylvania Bldg., Philadelphia, Pa.

Giddings, Ralph, 18th St. and North Blvd., College Point, N. Y. (H. D. Best Co.)

Gilman, Charles, 50 Church St., New York City. (Massey Concrete Products Corp.)

Gleason, Robert W., United Bank Bldg., Paterson, N. J. (John W. Ferguson Co.)

*Glens Falls Portland Cement Co., 205 Lower Warren St., Glens Falls, N. Y. (G. F. Boyle.)

Godfrey, Edward, Monongahela Bank Bldg., Pittsburgh, Pa.

Goldbeck, A. T., 515 14th St., Washington, D. C. (Bureau of Public Roads.)

GOLDIE MFG. CO., Trenton Ave. and P. R. R., Wilkinsburg, Pa. (Wm. Goldie, Jr.)
GONNERMAN, H. F., 1501 S. Garfield Ave., Alhambra, Cal.
GOODMAN, C. R., P. O. Box 757, Orange, Tex.
GOTTSCHALK, L. F., Columbus, Neb.
GOW, CHARLES R., 166 Devonshire St., Boston, Mass.
GRAM, LEWIS M., 912 Oakland Ave., Ann Arbor, Mich. (University of Michigan.)
GRAY CONCRETE CO., Thomasville, N. C. (F. B. Gray.)
GRAY CONSTRUCTION CO., LTD., J. V., 541 Queen St. E., Toronto, Ont. (R. J. Fuller.)
GRAY, HOWARD ALLISON, 15 Beacon St., Boston, Mass. (Fay, Spofford & Thorndike.)
GREEN, JR., J. SINGLETON, 28 Moss Lane West, Brooks' Bar, Manchester, Eng.
GREENMAN, RUSSELL S., State Engineer's Dept., Albany, N. Y.
GRIMM, PRESTON C., 304 N. 9th St., Apollo, Pa.
GUARANTEE CONSTRUCTION CO., 140 Cedar St., New York, N. Y. (Edward Burns.)
HAGGERT, C. N., 331 4th Ave., Pittsburgh, Pa.
HALL, EDWIN C., 1037 45th St., Milwaukee, Wis.
HALL, QUINCY A., 212 Metropolitan Bank Bldg., Minneapolis, Minn.
HAMILTON, CHARLES T., 510 Hastings St. W., Vancouver, B. C.
HAMMER, EDMUND WALTER, 74 Lincoln Pl., E. Rutherford, N. J. (Thompson-Starrett Co.)
HAMMILL, HAROLD B., 42 Portsmouth Rd., Piedmont, Oakland, Cal.
HARDY, RICHARD, 1011 James Bldg., Chattanooga, Tenn. (Dixie Portland Cement Co.)
HARGEN, STANLEY, 133 Rutland Road, Brooklyn, N. Y. (Standard Oil Co.)
HARMS, H. J., 91 Quai Courbevoie, Courbevoie, Seine, France.
HARRIS, E. C., Wichita Falls, Tex. (Potts & Prentice.)
HARRIS, WALLACE R., 5235 Cornell Ave., Chicago, Ill. (International Trade Press, Inc.)
HART, R. E., Watkins Institute Bldg., Nashville, Tenn.
HART, WALTER E., 642 Metropolitan Bank Bldg., Minneapolis, Minn. (Portland Cement Association.)
HATT, WILLIAM KENDRICK, Purdue University, Lafayette, Ind.
HATTON, HOLMES & ANTHONY, 323 Masonic Temple, Cedar Rapids, Ia.
HAVLIK, R. F., Mooseheart, Ill.
HAY, FRANCIS H., 836 N. Andrews Blvd., Los Angeles, Cal. (The Foundation Co.)
HAYES, J. E. ENGINEERING CORP., Tientsin, China.
HAYWARD, HARRISON W., Mass. Inst. of Technology, Cambridge, Mass.
HEALY, CLARENCE, LINDE-GRIFFITH CO., Newark, N. J.
HEINE CHIMNEY CO., 123 W. Madison Ave., Chicago, Ill. (Eric Plagwit.)

Heller-Murray Co., 222 W. Rayen St., Youngstown, Ohio. (A. H. Heller.)
Hertzberg, Charles S. L., 239 Confederation Life Bldg., Toronto, Ont.
Hewett, W. S., 530 Metropolitan Bank Bldg., Minneappolis, Minn.
Heyworth, James O., Harvester Bldg., Chicago, Ill.
Hibbs, Manton E., 1423 N. 15th St., Philadelphia, Pa.
Hickok Construction Co., 1219 Flour Exchange, Minneapolis, Minn. (H. M. Hickok.)
Higginson, Clarence H., 18 East 41st St., New York, N. Y.
Hildreth & Co., 15 Broad St., New York City.
Hilker Supply Co., 16th and State Sts., Granite City, Ill. (E. W. Hilker.)
Hirschberg, Walter P., 218 Stephenson Bldg., Milwaukee, Wis. (Federal Engineering Co.)
Hitchcock, Frank A., Washington, D. C. (Bureau of Standards.)
Hoeffer & Co., Chamber of Commerce Bldg., Chicago, Ill. (Alexander C. Warren.)
Hoff, J. Haakon, 208 S. LaSalle St., Chicago, Ill. (American Bridge Co.)
Hoff, Olaf, 149 Broadway, New York City.
Hogentogler, C. A., 1819 M St.. N. W., Washington, D. C. (U. S. Bureau of Public Roads.)
Holabird & Roche, 104 So. Michigan Ave., Chicago, Ill. (E. A. Renwick.)
Hollister, S. C., 531 Land Title Bldg., Philadelephia, Pa.
Holmes, Francis, 248 Lambton Quay, Wellington, New Zealand.
Hool, George A., College Hills, Madison, Wis. (University of Wisconsin.)
Hoover, A. P., 52 Vanderbilt Ave., New York City.
Hope, B. C., Canton, N. C. (Champion Fiber Co.)
Hope Engineering Co., Harry M., 185 Devonshire St., Boston, Mass. (F. B. Galaher.)
Hopkins, Ralph Z., 2576 Hurlbut Ave., Detroit, Mich. Hudson Motor Car Co.)
Horn, H. M., 17 Battery Place, New York City.
Horner, Wesley W., 300 City Hall, St. Louis, Mo.
Horr, George E., 244 Madison Ave., New York City. (Turner Construction Co.)
Houk, Howard H., 234 L. Guerrero, Manila, Philippine Islands.
Howe, C. D., The Whelan Bldg., Port Arthur, Ont.
Howe, H. N., 76 Porter Bldg., Memphis, Tenn.
Howe & Peters, 24 California St., San Francisco, Cal. (Walter C. Howe.)
Howes, Benjamin A., 70 Fifth Ave., New York City.
Hoyer-Ellefsen, P. O. Box 463, Kristiania, Norway. (August Gundersen.)
Hoyt, John T. Noye, Marquette Bldg., Detroit, Mich. (Albert Kahn.)
Hoyt, W. A., Altoona, Pa.
Hubbard, Fred, 369 Frick Ave., Pittsburgh, Pa. (Carnegie Land Co.)
Hudson, R. J. H., Thorneycroft, Cambridge Road, Sidcup, Kent, England.

HUEBER BROS., 243 Baker Ave., Syracuse, N. Y. (B. V. Hueber.)

HUGHES, R. G., 152 Market St., Paterson, N. J. (John W. Ferguson Co.)

HULBERT, J. COWAN, 300 Caxton House, Westminster, S. W. 1, London, England.

HUME PIPE CO. (SOUTH AFRICA), LTD., National Bank Bldgs., Simmonds St., Johannesburg, South Africa. (Walter Wolstenholme.)

HUMPHREY, D. S., Euclid Beach Park, Cleveland, Ohio. (The Humphrey Co.)

HUMPHREY, RICHARD L., 805 Harrison Bldg., Philadelphia, Pa.

HURLBURT, R. W., 100 Jarvis St., Toronto, Ont.

HURLBUT, CHARLES C., 101 Park Ave., New York City, N. Y.

*HURON PORTLAND CEMENT CO., Ford Bldg., Detroit, Mich.

HUTCHINSON, G. W., East Morgan St., Raleigh, N. C. (North Carolina State Highway Comm.)

HYDRO-ELECTRIC POWER CO., 190 University Ave., Toronto, Ont.

IDEAL CONCRETE CONST. CO., 455 Rowell Ave., Joliet, Ill. (Gilbert Cooper.)

ILLINOIS IMPROVEMENT & CONSTRUCTION CO., 208 S. LaSalle St., Chicago, Ill. (E. H. Kuttner.)

ILLINOIS STEEL CO., Chicago, Ill. (T. J. Hyman.)

INDEPENDENT GRAVEL CO., Frisco Bldg., Joplin, Mo. (S. A. Fones.)

INDEPENDENT CONCRETE PIPE CO., 201 N. West St., Indianapolis, Ind. (Howard Schurman.)

INDUSTRIAL ENGINEERING CO., 30 Church St., New York City, N. Y. (D. Traver Miller.)

INGBERG, S. H., Bureau of Standards, Washington, D. C.

INGEMANSON, THURE W., 5944 W. Erie St., Austin Sta., Chicago, Ill.

*INLAND STEEL CO., First National Bank Bldg., Chicago, Ill. (G. H. Jones.)

INSLEY, WM. H., INSLEY MFG. CO., P. O. Box 167, Indianapolis, Ind.

*INSLEY MFG. CO., P. O. Box 167, Indianapolis, Ind. (Wm. H. Insley.)

INTERNATIONAL CEMENT CORPORATION, Havana, Cuba. (H. Martin Rivero.)

IRWIN, ORLANDO W., 1128 Ford Ave., Youngstown, O. (Truscon Steel Co.)

JEWETT, JOHN Y., Administration Bldg., Balboa Park, San Diego, Cal.

JOHNSON, ALGOT F., 809 1st Nat'l Soo Line Bldg., Minneapolis, Minn.

JOHNSON, A. L., Mutual Life Bldg., Buffalo, N. Y. (Corrugated Bar Co.)

JOHNSON, ERICK B., 803 8th Ave. South, Jamestown, N. Dak.

JOHNSON, LEWIS J., Harvard University, Cambridge, Mass.

JOHNSON, N. C., 149 Broadway, New York City.

JOHNSON, T. H., 319 Iowa Bldg., Sioux City, Ia.

JOHNSTON, ROBERT S., 3604 McKinley St., Washington, D. C. (Bureau of Standards.)

JONES CONSTRUCTION CO., H. N., 208 Moore Bldg., San Antonio, Tex. (H. N. Jones.)

JUNGCLAUS CO., WM. T., 825 Massachusetts Ave., Indianapolis, Ind. (F. W. Jungclaus.)

KAHN, ALBERT, Marquette Bldg., Detroit, Mich.

Kahn, Gustave, Youngstown, O. (Truscon Steel Co.)

Kaiser, B. J., 619 N. St. Clair St., Pittsburgh, Pa. (Bernard H. Prack.)

Kalman Co., Paul J., Merchants Natl. Bank Bldg., St. Paul, Minn. (L. O. Helgesen.)

*Kalman Co., Paul J., 22 W. Monroe St., Chicago, Ill. (G. E. Routh.)

*Kalman Co., Paul J., 22 W. Monroe St., Chicago, Ill. (W. E. White.)

Kalman Floor Co., The, 22 W. Monroe St., Chicago, Ill. (C. E. Cooke.)

Kapp, P. A., 707 W. College Ave., State College, Pa. (Penn State College.)

Kattele, Walter R., Western Electric Co.

Kearney, E. N., P. O. Box 206, New Orleans, La.

*Kearns Constr. Co., 153 Milk St., Boston, Mass. (W. F. Kearns.)

Kelley, Frederick W., 126 State St., Albany, N. Y. (Hildenberg Cement Co.)

Kelly Construction Co., P. H., 1713 Sansom St., Philadelphia, Pa. (P. H. Kelly.)

Kelty,* Emer G., 122 N. 51st St., Philadelphia, Pa. (Consolidated Expanded Metal Co.)

Kent, Cecil Frederick, 10 Burnley Road, Dollis Hill, London, N. W. 10, England. (Winn & Kent.)

Kerr, Horace D., Frederick Bldg., Cleveland, O. (Nichols-Moore Co.)

Kikuchi, Aitaro, Toa Cement Co., Ltd., Amagasaki, near Osaka, Japan.

Kingsbury, C. T., 216 Woodward Bldg., Washington, D. C. (Rosslyn Steel & Cement Co.)

Kinney, William M., 111 W. Washington St., Chicago, Ill. (Portland Cement Assn.)

Klinger, W. A., Warnock Bldg., Sioux City, Ia.

*Knickerbocker Portland Cement Co., 342 Madison Ave., New York City. (A. D. Naylor.)

Knoph, Olaf, Munchsgate F., Kristiania, Norway.

Knudsen, Ax. M. and S. L. Sorensen, Vesterbrogade 13, Copenhagen, Denmark. (N. J. Nielsen.)

*Koehring Company, 31st and Concordia Ave., Milwaukee, Wis. (P. Koehring.)

*Koehring Company, 4940 N. 8th St., Philadelphia, Pa. (George A. Sherron.)

Komuro, Mangoro (Iwaki Cement & Co., Ltd.), Yotsu Kuracho, Fukushi, Maken, Japan.

*Kosmos Portland Cement Co., 614 Paul Revere Bldg., Louisville, Ky. (O. N. Clarke.)

Kraft, Rudolph George, 171 Woodward Ave., Buffalo, N. Y. (Turner Const. Co.)

Krause, G. E., P. O. Box 114, Juneau, Alaska.

Krause, Mark C., 120 West 4th St., Williamsport, Pa.

Krier, George H., 814 East 94th St., Brooklyn, N. Y.

Kvitrud, I., 2001 7th St. South, Minneapolis, Minn.

*Laclede Steel Co. (W. L. Allen), 1317 Arcade Bldg., St. Louis, Mo.

Lagaard, M. B., Experimental Engineering Bldg., Minneapolis, Minn. (University of Minnesota.)

Lake, Simon, Milford, Conn.

*Lakewood Engineering Co., 11723 Detroit Ave., Cleveland, Ohio. (W. N. Keiser.)

Lamb Co., Robert E., 843 North 19th St., Philadelphia, Pa. (Robert E. Lamb.)

Lambert, Walter E., 2028 Lincoln St., Evanston, Ill.

Lambie, J. Edward, 299 Broadway, New York, N. Y. Lambie Concrete House Corp.)

Lambie, Joseph S., Parkman Blvd., Pittsburgh, Pa. (University of Pittsburgh.)

Lander, R. S. (Sherman Concrete Pipe Co. Burwell Bldg., Knoxville, Tenn.

Landor, Edward J., 634 Renekert Bldg., Canton, Ohio.

Lane, H. A., Baltimore and Ohio Central Bldg., Baltimore, Md. (Baltimore & Ohio Railroad Co.)

*Lanquist & Illsley Co., 1100 N. Clark St., Chicago, Ill. (A. Lanquist.)

Lapham, John R., 1829 G St., N. W., Washington, D. C. (George Washington Univ.)

Larson, Reuben Lawrence (Anderson, Meyer & Co., Ltd.), Shanghai, China.

"La Tolteca," Cia de Cemento, Portland, S. A., Independencia 8, P. O. Box 233, Mexico, D. F., Mexico. (G. H. E. Vivian.)

Lavigne, Ernest T., 280 St. Cyrille St., Quebec, P. Q. (Quebec Provincial Dept. of Public Works & Labor.)

*Lawrence Portland Cement Co., Northampton, Pa. (J. S. Van Middlesworth, 302 Broadway, New York City.)

Lea, William S., 809 New Birks Bldg., Phillips Square, Montreal, Que. (R. S. & W. S. Lea.)

Leffler, Ralph R., 4850 Hutchinson St., Chicago, Ill.

Lehigh Portland Cement Co., Young Bldg., Allentown, Pa.

*Leonard Construction Co., 375 Wabash Ave., Chicago, Ill. (Clifford M. Leonard.)

Leonard, John B., 57 Post St., San Francisco, Cal.

Lesley, Robert W., 611 Pennsylvania Bldg., Philadelphia, Pa.

*Levering & Garrigues Co., 552 West 23rd St., New York City. (C. B. Wigton.)

*Ley, Fred T., & Co., 495 Main St., Springfield, Mass. (Raymond K. Turner.)

Libberton, J. H., 25 Broad St., New York City. (General Chemical Co.)

Lieberman & Hein, 190 N. State St., Chicago, Ill. (C. Lieberman.)

Lind, Peter & Co., 2 Central Bldg., Westminster, London, S. W. 1, England.

Lindau, A. E., Corrugated Bar Co., Buffalo, N. Y.

LINDSLEY Co., C. E., 888 Clinton Ave., Irvington, N. J. (C. E. Lindsley.)

LOCK JOINT PIPE Co., P. O. Box 21, Ampere, N. J. (A. M. Hirsh.)

LOCKE, CLYDE E., 905 Ellicott Sq., Buffalo, N. Y. (A. E. Baxter Eng. Co.)

LOCKHARDT, BYRNE Co., IND., 512 Fifth Ave., New York City, N. Y. (William F. Lockhardt.)

LOCKWOOD, GREENE & Co., 60 Federal St., Boston, Mass. (Library.)

LOGEMAN, R. T., 208 S. LaSalle St., Chicago, Ill. (American Bridge Co.)

LONEY, NEIL M., 51 Wall St., New York City. (Thompson-Starrett Co.)

LONG, M. A., 1523 Munsey Bldg., Baltimore, Md. (The M. A. Long Co.)

LORD, ARTHUR R., 140 S. Dearborn St., Chicago, Ill. (Lord Eng. Co.)

LOS ANGELES CONCRETE TILE Co., 432 I. W. Hellman Bldg., Los Angeles, Cal. (Harry Soderberg.)

LOVE, GEORGE C., Newport News, Va. (Old Dominion Land Co.)

LOVE, HARRY J., 933 Leader-News Bldg., Cleveland, Ohio. (Nat. Slag Assn.)

LOVIS, ANDREW M., Room 511, State House, Boston, Mass.

LOWELL, JOHN W., 208 LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)

LOWRY, JR., JOHN, 171 Madison Ave., New York, N. Y.

LUNDOFF-BICKNELL Co., THE, 5716 Euclid Ave., Cleveland, O. (C. W. Lundoff.)

LUTEN, DANIEL B., 1056 Lemeke Annex, Indianapolis, Ind.

LYNAM, C. G., Public Works Dept., Bagdad, Mesopotamia.

MCBURNEY, J. W., 6 Rockwell Bldg., Cleveland, Ohio. (Cleveland Board of Education.)

MCCLATCHY, JOHN H., 848 Land Title Bldg., Philadelphia, Pa.

MCCLENDON & PURNELL, Mineral Wells, Texas. (William McClendon.)

MCCREE, A. A., 712 Merchants Bank Bldg., St. Paul, Minn. (McCree Moos & Co.)

MCCULLOUGH, F. M., Carnegie Institute of Technology, Pittsburgh, Pa.

MCDANIEL, ALLEN B., 1729 Q St., N. W., Washington, D. C.

MACFARLAND, DAVID H., 30 Broad St., New York City.

MCGREGOR, J. D., Union Sugar Co., Betteravia, Cal.

MCINTYRE, WILLIAM A., 809 Flanders Bldg., Philadelphia, Pa. (Atlas Portland Cement Co.)

MCKIBBEN, FRANK P., Schenectady, N. Y. (Union College.)

MCKINSTRY, ROSS W., Room 1012 Kimball Bldg., Chicago, Ill. (Bernard H. Prack.)

MCLACHLAN, PETER, 31 Church St., Warrington, England.

MCLAREN, ERIC J. R., Engineering Dept., Auckland University, Symonds St., Auckland, N. Z.

MCLEOD, WILLIAM, Balgownie Ave., Bonville, Wanganui, New Zealand.

MCMILLAN, FRANKLIN R., 628 Metropolitan Bank Bldg., Minneapolis, Minn. (Shenehon & Meyer.)

MCRAE STEEL Co., 16 McGraw Bldg., Detroit, Mich. (William Corman.

MACATEE, W. L., & SONS, Austin and Commerce Sts., Houston, Tex.

MACHNER, H. FRANK, 282 Rua Floriano Peixoto, Recife, Pernambuco, Brazil, S. A.

MACK, THOMAS, 122 S. Michigan Blvd., Peoples Gas Bldg., Chicago, Ill.

MADISON, JAMES T., Dept. of State Roads and Highways, Frankfort, Ky.

MAIN, CHARLES T., 201 Devonshire St., Boston, Mass.

*MALMED, A. T., 18 S. 7th St., Philadelphia, Pa. (A. T. Malmed Co.)

MALONEY OIL AND MFG. CO., Scranton, Pa. (W. M. Collins.)

MANITOBA, UNIVERSITY OF, Sherbrooke and Portage Sts., Winnipeg, Man. (J. N. Finlayson.)

MANTICA, ALBERT J., 607 Dickinson Bldg., Norfolk, Va. (Truscon Steel Co.)

MARBLE, WILLIAM O., 508 London Bldg., Vancouver, B. C. (Hodgson, King & Marble.)

MARQUETTE CEMENT MFG. CO., Marquette Bldg., Chicago, Ill. (T. G. Dickinson.)

MARTIN, EDGAR D., 104 S. Michigan Ave., Chicago, Ill.

MARTIN, EVAN S., 16 Saulter St., Toronto, Ont. (James A. Wickett, Ltd.)

MASSEY CONCRETE PRODUCTS CORP., Peoples Gas Bldg., Chicago, Ill. (Paul Kirscher.)

MASTER BUILDERS CO., THE, 1836 Euclid Ave., Cleveland, Ohio. (S. W. Flesheim.)

MASTRANGELO, M. J., 369 Hanover St., Boston, Mass.

MAYNARD, ARTHUR J., Mass. State Farm, State Farm, Mass.

MAZUR, ISADORE, 444 Virginia Ave., Indianapolis, Ind. (Truscon Steel Co.)

MEAD, C. A., 165 Wildwood Ave., Upper Montclair, N. J.

MEADERS, ERNEST LAMAR, McComb, Miss. (E. L. Meaders Co.)

MERCHANT, ARCHIE W., 728 Hospital Trust Bldg., Providence, R. I. (William & Merchant, Inc.)

MERIWETHER, COLEMAN, Louisville, Ky. (Meriwether Pressure Pipe Co.)

MESSEY, LAUREL, Commerce Bldg., Ash and George Sts., Sydney, Australia.

METCALF & EDDY, 14 Beacon St., Boston, Mass. (Frank A. Marston.)

MEYER, C. LOUIS, 608 Omaha National Bank Bldg., Omaha, Neb. (Concrete Engr. Co.)

MEYER, MORRISON & CO., 299 Broadway, New York City.

MICHIGAN PORTLAND CEMENT CO., Chelsea, Mich. (G. S. Potter, Jr.)

MICHIGAN UNIVERSITY LIBRARY, Ann Arbor, Mich.

MILLER, CHARLES R., CO., INC., 556 Susette St., Memphis, Tenn.

MILLIKIN, M. M., 708 New Zealand Insurance Bldg., Queen St., Auckland, New Zealand.

MINGLE, J. G., 540 N. Meridian St., Indianapolis, Ind.

MITCHELL, JAMES, 76 Montgomery St., Jersey City, N. J.

MITCHELL, NOLAN D., 220 Beech St. South, Clarendon, Va. (U. S. Government, Treasury Dept.)

Mitchell, Rudolph, Box 271, Port Arthur, Tex. (The Texas Co.)

Mohler, John D., Court House, St. Joseph, Mo. (Highway Engineer, Buchanan County.)

Monks & Johnson, 99 Chauncy St., Boston, Mass. (John R. Nichols.)

Monolithic Hollow Concrete Form Corp., 326-330 Pacific Finance Bldg., Los Angeles, Cal. (C. L. Desenberg.)

Morris, Clyde T., Ohio State Univ., Columbus, Ohio.

Morris, Lloyd M., 428 W. College Ave., State College, Pa. (Pennsylvania State College.)

Morrison, R. L., 4717 First Ave., Birmingham, Ala. (Concrete Products Co.)

Morrow, David W., 4500 Euclid Ave., Cleveland, Ohio.

Morssen, C. M., 37 Belmont St., Montreal, Que.

Moses, Frederick W., 10 Weybossett St., Providence, R. I. (Fireman Insurance Co.)

Moyer, Albert, 350 Madison Ave., New York. (Vulcanite Portland Cement Co.)

Mueller, Harold P., Palladium Bldg., Richmond, Ind. (John W. Mueller Co.)

Mueller, J. W., Palladium Bldg., Richmond, Ind.

Muller, Jack A., Pedro Miguel, Canal Zone. (Panama Canal.)

Mumm, Jr., Hans, 1009 Gasco Bldg., Portland, Ore. (Portland Cement Assn.)

Mylchrest, Geo. Lewis, 60 Prospect St., Hartford, Conn. (Buck & Sheldon, Inc.)

National Fireproofing Co., Flatiron Bldg., New York City. (P. Bevier.)

National Lime Association, 918 G. St., N. W., Washington, D. C.

*Nassau Sand & Gravel Co., 949 Broadway, New York City. (W. J. Timberman.)

*Nazareth Portland Cement Co., Nazareth, Pa. (J. A. Horner.)

Needham, Egbert S., Pinners Hall, Old Broad St., London, E. C. 2, England.

Nelson, John A., 1800 Eleventh St., Rock Island, Ill. (Cut Stone Co.)

Neuffer, George T., 509 W. Second St., Dayton, Ohio.

*Newaygo Portland Cement Co., Newaygo, Mich. (J. D. John.)

New Jersey Zinc Co., Palmerton, Pa. (Works Library.)

Newman, Stanley O., 47 Melrose Ave., East Orange, N. J.

Nichols, Charles Eliot, 147 Milk St., Boston, Mass. (Stone & Webster, Inc.)

Nicholson, Jr., John, 2735 Prospect Ave., Cleveland, Ohio.

Noble & Co., R. E., 624 Sacramento St., San Francisco, Cal. (Theodore P. Dresser, Jr.)

Noice, Blaine, 1326 Washington Bldg., Los Angeles, Cal.

Noonan, W. H., Metropole Bldg., Halifax, Nova Scotia.

Northwestern Expanded Metal Co., 37 W. Van Buren St., Chicago, Ill. (H. W. Fottee.)

NORTHWESTERN STATES PORTLAND CEMENT CO., Mason City, Ia. (W. Cowhan.)

NORTON, EDGAR W., 390 Main St., Worcester, Mass.

NOVELLE, GUSTAVO, Avenida del Hipodromo, Guatemala, Guatemala, C. A.

OAKEY, CHARLES W., P. O. Box 375, Sauk City, Wis.

OEHMANN, JOHN W., Room 110 District Bldg., Washington, D. C. (Prin. Asst. Inspector of Buildings, D. C.)

OESTERBLOM, I., British American Machinery Co., Lalbazaar St., Calcutta, India.

OGDEN PORTLAND CEMENT CO., Ogden, Utah. (R. C. Briscoe.)

OKUBO, TOSHIYUKI (Truscon Steel Co. of Japan), Curakucho Kojimachi, Tokyo, Japan.

OLDER, CLIFFORD, State Highway Comm., Springfield, Ill.

OLESON, OLE K., 822 Pendido St., New Orleans, La.

OLSEN, EINAR HELMERS, Kong Oscarsgate 45, Bergen, Norway.

ORD, WILLIAM, Lakewood Eng., Co., Cleveland, Ohio.

ORR, JOHN B., 6th St., Miami, Fla.

OSAWA, ICHIRO, P. O. Box 157, Urbana, Ill. (University of Illinois.)

*OTTAWA SILICA CO., Ottawa, Ill. (C. B. Herring.)

PAGET, A. MAXWELL, 224 N. Front St., Wormleysburg, Pa.

PAINTER, W. S., Engineers Club, New York City, N. Y.

PARKER, FRANK S., 44 Court St., Brooklyn, N. Y.

PARRY, CHARLES, 1512 Walnut St., Philadelphia, Pa.

PATTILLO, JAMES N., 1716 W. 42nd St., Los Angeles, Cal.

PEABODY, DEAN, JR., Room 1-302 Mass. Institute of Technology, Cambridge, Mass.

PEARSE, LANGDON, 910 S. Michigan Ave., Chicago, Ill. (Sanitary District of Chicago.)

PEARSON, J. C., Bureau of Standards, Washington, D. C.

PEASE, B. S., 208 S. LaSalle St., Chicago, Ill. (Am. Steel & Wire Co.)

*PEERLESS PORTLAND CEMENT CO., Union City, Mich. (William M. Hatch.)

*PENN-ALLEN PORTLAND CEMENT CO., Widener Bldg., Allentown, Pa. (W. E. Eidel.)

*PENNSYLVANIA CEMENT CO., 30 E. 42nd St., New York City. (Wm. Beach.)

PENNSYLVANIA STATE HIGHWAY DEPT., Harrisburg, Pa. (Willis Whithed.)

PERROT, EMILE G., Boyertown Bldg., 1211 Arch St., Philadelphia, Pa.

PERROTT, LESLIE M., Architect, 243 Collins St., Melbourne, Victoria, Australia.)

PETROLEUM HEAT & POWER CO., 100 Boylston St., Boston, Mass. (C. W. Stancliffe.)

PITTSBURGH TESTING LABORATORY, 7th and Bedford Ave., Pittsburgh, Pa. (J. M. Bailey.)

PLAGWIT, ERIC, 123 W. Madison St., Chicago, Ill.

PLANO CEMENT PRODUCTS CO., Box B., Plano, Ill. (Chas. A. Steward.)

PLUMER, H. E., 22 Ellicott Square, Buffalo, N. Y.

PONSFORD, H. J., 914 East Missouri St., El Paso, Tex.

Porter, J. M., Easton, Pa.

*Portland Cement Assn., 111 W. Washington St., Chicago, Ill.

Post Mfg. Co., The, Care of Kent Machine Co., Kent, Ohio. (F. R. Brantigam.)

Powell, Lester H., 914 Monadnock Block, Chicago, Ill. (Wells Bros. Const. Co.)

Powers & Son, Eugene S., 315 S. 15th St., Philadelphia, Pa. (E. S. Powers.)

Prack, Bernard H., 50 Bay St., Toronto, Ont. (Prack & Perrin.)

Pre-Cast Concrete Co., 610 Bulletin Bldg., Philadelphia, Pa. (Chas. D. Watson.)

Prindiville Co., J. J., Framingham, Mass. (J. J. Prindiville.)

Pritchard, Edwin M., 251 Vreeland Ave., Nutley, N. J.

Pucci, Angelo, 325 Central Park, Rochester, N. Y.

Purves, John, 914 Monadnock Block, Chicago, Ill. (Wells Bros. Const. Co.)

Quebec Dept. of Public Works & Labor, Government Bldgs., Quebec, P. Q. (Ivan E. Vallee.)

Quebec Dept. of Roads, Parliament Bldgs., Quebec, P. Q. (B. Michaud.)

Rader, B. H., Conway Bldg., Chicago, Ill. (Lehigh Portland Cement Co.)

Randall, Frank A., 19 S. LaSalle St., Chicago, Ill. (Berlin, Swerm & Randall.)

Ransohoff, V., Ideal Concrete Machinery Co., Cincinnati, Ohio.

*Ransome Concrete Machinery Co., Dunellen, N. J. (A. P. Robinson.)

*Ransome Concrete Machinery Co., Dunellen, N. J. (A. P. Robinson.)

Ransome, A. W., Monadnock Bldg., San Francisco, Cal. (Blaw-Knox Co.)

*Raymond Concrete Pile Co., 140 Cedar St., New York City. (Paul D. Case.)

Reed Co., William T., 201 Devonshire St., Boston, Mass. (William T. Reed.)

Reubell, J. C., 1000 W. Washington Ave., N. Little Rock, Ark. (Margranite Products Co.)

Reynvaan, A. J., 423 Biddle Ave., Wilkinsburg, Pa.

Rheinstein & Hass, I&3., 21 E. 40th St., New York City. (A. Rheinstein.)

Rhett, Albert H., Room 709, 320 Fifth Ave., New York City. (Toch Bros.)

Rib-Stone Concrete Corp., 2-3 Chamber of Commerce Bldg., Batavia, N. Y. (George E. Priest.)

Rice, James, P. O. Box 10, Forest Hills, L. I.

Richards II, Clarence E., 584 E. Broad St., Columbus, Ohio. (Richards, McCarty, & Bulford.)

Richardson, J. R., Madera, Cal.

Richardson-Jones, Harry, 1726 Anapuni St., Honolulu, Hawaii.

Richart, Frank E., University of Illinois, Urbana, Ill.

Richmond, Knight C., 10 Weybossett St., Providence, R. I.

RIESCHE, ROBERT H., 306 Trimble Blk., Sioux City, Ia. (Riesche & Sanborn.)

RITTER, LOUIS E., 140 So. Dearborn St., Chicago, Ill. (Ritter & Matt.)

ROBINSON, ALBERT FOWLER, Room 1033, Railway Exchange Bldg., Chicago, Ill. (A., T. & S. F. R. R. System.)

ROBINSON, C. C., 1004-5 Times-Despatch Bldg., Richmond, Va. (Chas. M. Robinson.)

ROBINSON & CO., INC., Dwight P., 125 E. 46th St., New York City. (M. E. Thomas.)

ROGERS, FLOYD, Newton, Iowa.

ROGERS, FRANKLIN, 38 So. Dearborn St., Chicago, Ill. (Lockwood, Greene & Co.)

ROGERS, J. S., Co., Moorestown, N. J. (C. R. Rogers.)

ROGERS, WARREN A. (Blystone Mfg. Co.), Cambridge Springs, Pa.

ROGOW, SYDNEY, Room 1117, 80 Wall St., New York City.

ROLLINS, JAMES W., 6 Beacon St., Boston, Mass. (Holbrook, Cabot & Rollins.)

ROLLINS, J. FRANK, 802 Owen Bldg., Detroit, Mich. (Michigan Architect & Engineer.)

ROOS CO., THE H. W., 2036-46 Dana Ave., Cincinnati Ohio. (H. W. Roos.).

ROYAL SWEDISH BOARD OF WATERFALLS, Regeringsgatan 45, Stockholm, 3, Sweden. (Axel Ekwall.)

RYAN, R. F., Phillippi, W. Va.

RYAN, WILLIAM R., 49 Wall St., New York City. (Thompson-Starrett Co.)

RYLEY, E. G., 506-7 P. Burns Bldg., Calgary, Alberta, Canada. (Truscon Steel Co.)

SAFFORD, A. T., 66 Broadway, Lowell, Mass.

SAMPSON, GEORGE A., 83 Pembroke St., Newton, Mass. (Weston & Sampson.)

SANADA, K., Care of Civil Engineering Section, Engineering Dept., S. M. R. Co., Dairen, South Manchuria, China.

SANDSTROM, CHARLES O., 515 Reliance Bldg., Kansas, Mo.

*SANDUSKY CEMENT CO., 626 Engineers Bldg., Cleveland, O. (Wm. B. Newberry.)

SANTA CRUZ PORTLAND CEMENT CO., Davenport, Cal. (Llewellyn T. Bachman.)

SASSO, MAURO, 318 W. 55th St., Los Angeles, Cal. (Globe Grain & Milling Co.)

SAUM, IRVING R., 3218 Newark St., N. W., Washington, D. C. (Chas. H. Tompkins.)

SAUNDERS, W. L., 1323 Columbia Road, Washington, D. C. (Concrete Steel Co.)

SAURBREY, ALEXIS, 2112 Oliver Bldg., Pittsburgh, Pa. (Mellen-Stuart Co.)

SAVILLE, CHRISTOPHER JAMES, Portland, New South Wales, Australia. (Commonwealth Portland Cement Co.)

SCHAEFER BROS., 121 Powers Bldg., Rochester, N. Y. (Albert F. Schaefer.)

Schilling, Adolph, Haddon Heights, N. J. (Eastern Potash Corp.)
Schlyter, Ragner, State Testing Institute, "Statens Provningsanstalt," Stockholm, Sweden.
Schmidt, Paul S., 4500 Euclid Ave., Cleveland, Ohio.
Schofield, R. W., Whakatone, New Zealand.
Schouler Cement Construction Co., 154-6 Frelinghuysen Ave., Newark, N. J. (D. D. Schouler.)
Schuler, Albert V., R. F. D. No. 5, Royal Oak, Mich.
Schuster, K. R., 15 Park Row, New York, N. Y.
Schwalbe, William, 711 W. Springfield St., Urbana, Ill. (University of Illinois.)
Schwendener, Karl D., 1326 Washington Bldg., Los Angeles, Cal.
Scofield Engr. Construction Co., Wright-Callender Bldg., Los Angeles, Cal. (C. M. Scofield.)
Scott, J. R., Ross & Scott, Welland, Ont., Can.
Security Cement & Lime Co., Hagerstown, Md. (John Porter.)
Seelye, Elwyn E., 101 Park Ave., New York City.
Sewell, Col. John S., Muncie, Ind. (Maxon Furnace & Eng. Co.)
Shafer, Ivan O., 531 Seventh Ave., New York City. (Fleischmann Construction Co.)
Shallcross, J. B., 510-1900 Euclid Bldg., Cleveland, Ohio. (Western Waterproofing Co.)
Shank, J. R., 97 W. Tompkins St., Columbus, Ohio.
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